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Leveling device settings and transverse joint material testing for full depth precast panel bridge deck assembly

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LEVELING DEVICE SETTINGS & TRANSVERSE JOINT MATERIAL TESTING FOR FULL DEPTH PRECAST PANEL BRIDGE DECK ASSEMBLY

BY

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THESIS

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ABSTRACT

LEVELING DEVICE SETTINGS & TRANSVERSE JOINT MATERIAL TESTING FOR FULL DEPTH PRECAST PANEL BREIDGE DECK ASSEMBLY

By

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University of New Hampshire, May, 2013

Across the United States there are many deteriorating highway bridges that are in need of replacement. In 2013, the Federal Highway Administration estimated that one-fourth of the Nation's bridges are in need of rehabilitation, repair, or complete replacement. (FHWA, 2013) To address these needs, innovative techniques must be developed to reduce both lifecycle costs and construction times while increasing both the quality and safety of bridge rehabilitation projects.

This research aims to further develop a method of integrating structural analysis software, during the modular construction process, to predict the leveling device settings for precast prestressed panels to ensure the desired profile of the bridge deck is met. This research also includes the material testing of structural adhesives for use in full depth precast panel assembly. This research concludes that the integrated use of structural analysis software enables the accurate prediction of leveling screw device settings to replicate the desired profile of a full depth precast concrete bridge deck.

CHAPTER 1

1. INTODUCTION

Accelerated Bridge Construction is a bridge construction method that implements innovative techniques throughout the planning, design, and construction phases to improve safety, reduce cost, and shorten construction time. (FHWA, 2013) Accelerated Bridge Construction is a viable procedure for the rehabilitation or replacement of bridges throughout the United States.

1.1. Need for Research

With the enactment of the Federal Aid Highway Act of 1956, over 40,000 miles of the Interstate Highway System was to be built over the following fourteen years. (Weingroff, 1996) Along with these new roadways came the construction of a great number of bridges, on state and local roads throughout America, during the 1950's and 1960's. A majority of these bridges are still in use today. In 2010 roughly 200,000 of the nation's 600,000 interstate bridges were over 50 years old. (Shoup, et al., 2011) With the nation's continuously aging infrastructure, there is an immediate need for new advances in bridge construction technology in order to repair these bridges rapidly, safely, and in a cost friendly manner while still providing quality construction.

1.1.1. Quality of Highway Infrastructure

Producing high quality components is an important aspect of bridge construction. The quality of bridge components, especially concrete bridge decks, can be affected by the variability of onsite conditions. During the course of a construction sequence the materials and the crew are subject to variable weather conditions, time setbacks, unanticipated site conditions, variability in material performance, and equipment failures. This lack of a controlled fabrication environment negatively affects the quality and uniformity of cast-in-place concrete bridge decks. (Chavel, 2012) It is imperative to attempt to reduce the amount of variability in the construction process in order to improve the quality of construction and maximize the design life of rehabilitated and new structures. The quality of bridge components can be increased and controlled by casting the elements off site in a precast manufacturing facility that can provide optimal casting and curing conditions. (Chavel, 2012)

1.1.2. Traffic Congestion

With the growing population and aging, outdated infrastructure in America, traffic congestion is at an all-time high. It is estimated that "by the year 2020, ninety percent of the urban Interstate highways will be at or exceeding capacity." (FHWA, 2011) Traffic congestion can be caused by many things including undersized or outdated roadways, extensive detours, roadway closures, lane closures, alternate traffic patterns and increased road traffic. Most if not all of these occur during a construction project. Construction zones along with high traffic volumes have the potential to cause large traffic delays and backups. Bridge construction related traffic impact can be minimized by implementing well designed traffic patterns and routes as well as by shortening the duration of the construction process. (Harvey, 2011)

1.1.3. Safety

Human safety is a big concern on America's Highways. For the past decade, 43,000 people have died every year on America's highway system. 15,000 of those fatalities are directly related to the substandard condition of the roadway. (FHWA, 2011) Construction zones pose an even greater danger to motorists. These dangers include unfamiliar traffic patterns, objects and debris in the roadway, and movement of heavy equipment.

Road work is typically done with workers and motorists sharing the same roadway. Whether work is occurring in one of the roadway lanes or on the shoulder of the road, traffic must be routed around the work area. Lane closures, due to road construction, create hazards that are not present under normal driving conditions and can also cause heavy traffic backups and bottlenecking. Motorists are also forced to maneuver around heavy equipment and adverse road conditions.

Workers are also subject to a great deal of danger while working in construction zones. Workers are exposed to constant threats due to motorists, equipment, materials, and occupational hazards. During typical lane closures, one or two traffic lanes are closed for work to be done. This causes the workers to perform their duties in very close proximity to the motorist travel lanes.

1.2. Goals of Research

The goal of this research is to further develop and test an accelerated bridge construction procedure, focusing on the placement of full depth panels onto the girders, which can be used for the replacement or refurbishment of average size bridges locally and nationwide.

1.2.1. Research Overview

Full depth concrete panels have been used in non-composite construction since the 1960s and for full composite construction since 1973. (Hieber, et al., 2005) Precast panels are widely used for the replacement of deteriorated, cast-in-place concrete bridge decks as well as various new bridges. Full depth precast panels generally span the entire width of the bridge and are placed adjacent to one another in the longitudinal direction. (Chavel, 2012) One key issue with full depth concrete panels is the bearing of the panels onto the girders. In order to avoid differential bearing of the panels on the girders and ensure good performance, the panels should be leveled and bear evenly on the girders. (Hieber, et al., 2005) This research aims to develop a procedure to address this issue.

This accelerated bridge construction procedure will include the integration of structural analysis modeling to be used in the construction process. Structural Analysis modeling will be used in the calculation of the required leveling device lengths of full depth precast panels. Calculating and setting the leveling screw device lengths, prior to the placement of the panels, will reduce the time needed for panel installation.

This research also aims to develop a leveling device torqueing procedure to properly distribute the dead load of each precast panel to all girders in the system. Distributing the dead load of the panels amongst all girders in the system is required to avoid overloading any of the girders.

This research will also focus on the testing of structural adhesives to be used as a transverse joint material. The transverse joint of full depth precast panels is the joint in which two panels are joined together and, in the case of full width panels, runs transverse to the flow of traffic over the bridge. This transverse joint is considered a weak spot in the construction of full width precast panel bridge decks. (Chavel, 2012) This structural adhesive testing aims to identify a material for use in the transverse joint of full depth precast panels.

This accelerated bridge construction procedure aims to be applicable to any concrete deck replacement of a single-span bridge between fifty and one-hundred and twenty five feet long while conforming to all program guidelines set by the Federal Highway Administration's Highways for Life. (FHWA, 2011)

1.2.2. Highways for Life

Beginning in 2006, The Federal Highway Administration began receiving funding from the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) for the Highways for Life program. (Guy, 2007) This government program was put into place to aid in the advancement and acceptance of new technologies in America's highway system.

"The purpose of Highways for LIFE is to advance Long lasting highways using Innovative technologies and practices to accomplish Fast construction of Efficient and safe pavements and bridges, with the overall goal of Improving the Driving Experience for America." (Guy, 2007)

"Specifically, (Highways for LIFE) is focused on accelerating the adoption of innovations in the highway community." (FHWA, 2011) It wishes to focus on innovations in safety, traffic congestion, and quality of highway infrastructure. The Federal Highway Administration and Highways for Life plans to accomplish this by offering funding to transportation departments who prove to be using an innovative or progressive approach to their Federal-Aid highway project. The transportation departments must use innovative technologies, manufacturing processes, financing or contracting methods that meet the performance goals for safety, congestion relief and quality. (Guy, 2007)

1.2.3. Gilford Bridge Deck Replacement

Another purpose of this research is to aid in the bridge deck replacement of the Gilford Bridge, located in Gilford, NH. This research aims to further develop a method of accelerated bridge construction to be implemented in the deck replacement of the Gilford Bridge that conforms to all of the guidelines set forth by FHWA's Highways for LIFE program as well as reduce the upfront cost and duration of the construction process.

CHAPTER 2

2. BACKGROUND AND LITERATURE REVIEW

A great deal of research in the area of accelerated bridge construction has been completed both at the University of New Hampshire as well as other institutions around the country. These accelerated bridge construction techniques have been implemented in bridge rehabilitation projects as well.

2.1. University of New Hampshire Research

Research in the area of accelerated bridge construction has been ongoing at the University of New Hampshire since 2006. This research has been focused on developing a process for the replacement of a cast-in-place concrete bridge deck with pre cast full depth concrete panels. This research began with a post tensioning and sealant study. (Salzer, 2008) This was followed by a transverse joint configuration study in 2008. (Robert, 2011) In 2010, a study on the integration of finite element modeling in the construction process was conducted. (Pelletier, 2012) This research aims to address the concerns of these past research topics as well as develop new methods and processes.

2.1.1. Post tensioning and Sealant Study

In 2006, research was conducted to determine if the post tensioning process of concrete panels could be completed using a threaded rod system and staged construction techniques. The goal of the research was to post tension two independently cast panels together using a threaded rod system, then post tension a third panel, and all subsequent panels to the joined panels resulting in a fully composite panel system. (Salzer, 2008)

This research was followed by a sealant study to develop an effective process to seal and protect the post tensioning system from environmental effects. Research was also conducted to develop an efficient transverse joint configuration for the transfer of shear between adjacent panels.

The sealant study produced a process of sealing the transverse joints of each panel with a structural sealant. An injection process for the post tensioning ducts was also developed. This required the injection of a waterproof monomer, Methyl Methacrylate, into the post tensioning ducts after the post tensioning process is complete.

This research concluded that the use of the THREADBAR® post tensioning system from Dywidag Systems International, Inc. (DSI) was effective in achieving the compressive forces required for post tensioning throughout the system. (Salzer, 2008) Multiple panels were successfully joined, one after the other, using threaded post tensioning rods. It also concluded that "the differential deflection between adjacent panels for the tongue and groove transverse joint configuration was much less than that of the butt joint configuration. This was a result of the concrete-on-concrete shear transfer mechanism in the tongue and groove and the low modulus of the epoxy in the butt joint."

(Salzer, 2008) This indicated that the use of a tongue and groove transverse joint transferred shear more effectively than a butt joint configuration.

2.1.2. Transverse Joint Configuration Study

From 2008 to 2009, research was conducted on the performance of multiple transverse joint configurations in a post tensioned panel system. Research was conducted on four different transverse joint configurations including a butt joint, standard shear key, angular corrugated and round corrugated. Each of these transverse joint configurations was applied with a structural adhesive, with the exception of the shear key which was grouted with a high performance cement-based grout, and post tensioned together. The transverse joint configurations were then tested for their ability to transfer shear across the post tensioned sealed joints.

It was concluded that while the round corrugated transverse joint was the most efficient at transferring shear to adjacent panels it was also "by far the most difficult to fabricate." (Robert, 2011) It was concluded that the angular corrugated transverse joint configuration was also successful at transferring shear while being less difficult to fabricate. (Robert, 2011)

2.1.3. Rapid Bridge Deck Replacement Using Finite Element Modeling as a Construction Aid

From 2010 to 2012, research was conducted on the feasibility of using finite element modeling as a construction aid in accelerated bridge construction projects. The goal of this research was to reduce the construction time of bridge deck replacements using full depth precast concrete components. Full depth precast panels have been used for the construction of composite bridge decks since the 1970s. These panels have leveling devices cast into the panel which are used to obtain the desired profile of the panels. Traditionally, the panels are placed on the girders and then leveled using these devices. (Hieber, et al., 2005)

The key component of this research was "determining the lengths of the leveling screws and setting them to those lengths prior to placing the slabs on the girders." (Pelletier, 2012) This process would greatly reduce the installation time of the precast panels. This was accomplished by predicting girder deflections through the use of a finite element model, built in the SAP2000® structural analysis package produced by Computers and Structures, Inc. (CSI Berkeley, 2013)

A finite element model of the girder and panel system, that was located in the Structures Laboratory at the University of New Hampshire, was created in SAP2000®. This finite element model was built to replicate the girder system geometry of the lab model as well as the panel placement procedure.

The lab model for this research consisted of three, S15x42.9 steel girders spaced 7'-0" on center. The girders were connected with four C6x8.2 diaphragms, and rested on six supports with a clear span of twenty five feet. The girders were loaded with six, reinforced concrete, panels which measured 16'-0" x 3'-7.5" x 0'-8.5". These panels were cast with six leveling screws each. These six leveling screws, two per girder per panel, allowed for the control of the elevation profile of the top of the panel. Figure 1 shows the three girder and six panel lab model.

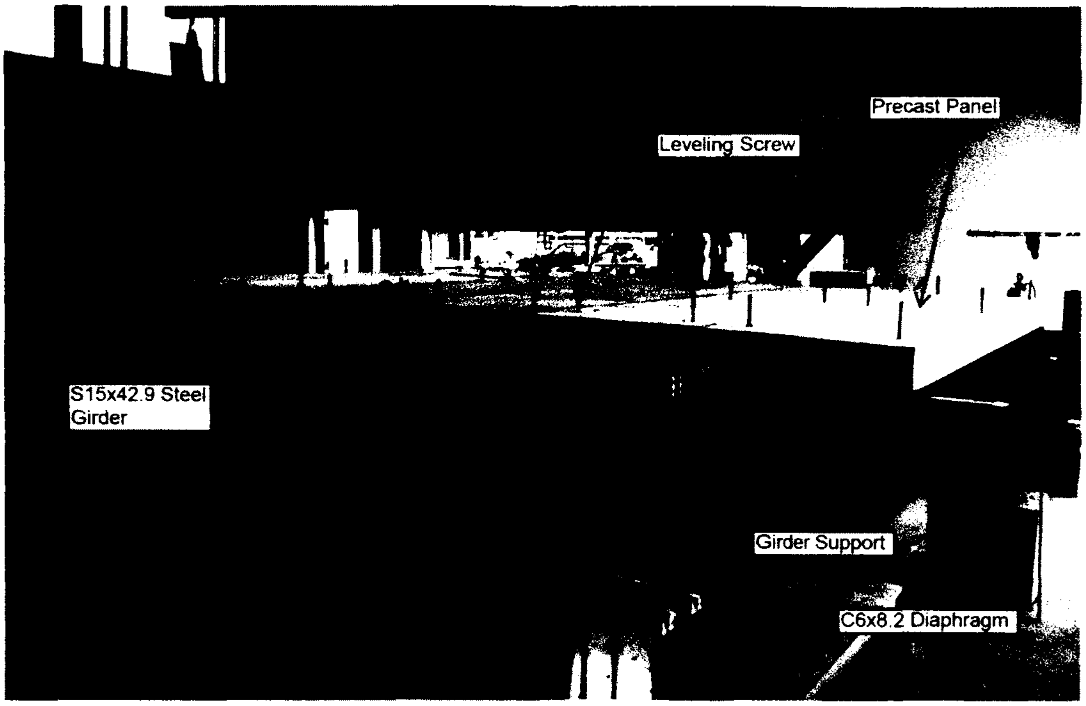

Figure 1: Three Girder and Six Panel Lab Model

(Photograph by: Justin Pelletier)

Calculating the required length of these leveling screws, to achieve the desired elevation profile of the top surface of the panels, was the main focus of this research. In order to predict the required length of the leveling screws, a finite element analysis model was built. This model replicated the properties of the lab model and was used to calculate the deflections of the steel girders under the dead load of the panels at each stage of assembly.

The assembly procedure outlined in this research was to place each panel, one after the other starting at one end, onto the girders. The dead loads of the panels were supported by the six leveling screws cast into each panel. The length of the leveling screws was calculated and set before each panel was placed. The leveling screw length was calculated using the geometry of the deflected girders from the SAP2000® model, under the full dead load of all of the panels, as well as the desired elevation profile of the top surface of the completed bridge deck.

A lab trial was performed to test this method of predicting the lengths of the leveling screws. The method "proved to be successful at predicting the required leveling screw lengths needed to make a smooth straight slab profile when the girders are under full dead load." (Pelletier, 2012)

Although the methods of this research were successful in predicting the lengths of the leveling screws, there were recommendations for future research. It was determined through this research that the use of a three girder lab model was not ideal. "If the leveling screws that contact the middle girder for one slab were set longer than the leveling screws for the other two girders, the slab would teeter on the middle girder." (Pelletier, 2012) Recommendations were made to conduct further research on a lab model consisting of four girders, which would eliminate this effect. Recommendations were also made to construct a lab model consisting of skewed panels to more accurately model the skew of the panels to be used in the bridge deck replacement of the Gilford Bridge. The modeling procedure used in this research, modeling objects with solid elements, "proved to be time consuming, tedious, and sensitive." (Pelletier, 2012) Recommendations were made to investigate the use of frame elements, instead of solid elements, to accelerate the model building and analysis process as well as simplify the updating of the SAP2000® model.

2.2. Accelerated Bridge Construction Deck Systems

One common technique, that conforms to the ideals of accelerated bridge construction very well, is the use of precast elements. (Hieber, et al., 2005) Precast elements generally entail a larger upfront construction cost, but present economy in the form of reduced construction time and labor costs. (Yamane, et al., 1998)

2.2.1. Full Depth Concrete Panels

Full depth precast panels generally span the entire width of the bridge. These panels are placed sequentially, one next to the other, to form a complete bridge deck. Full depth panels are typically cast with pockets for shear studs, or other mechanical connectors, to bond the panel to the girders. These pockets are grouted to develop composite action in the system. (Hieber, et al., 2005) Full depth panels use prestressed cable or mild steel reinforcement in the transverse direction and can also include longitudinal post tensioning to induce compression in the transverse joints of the panels and improve durability. (Hieber, et al., 2005) The installation of full depth concrete panels reduces construction time. This is because full depth panels eliminate the need to construct formwork and pour cast-in-place concrete. (Ralls, et al., 2005)

The most common form of damage in full depth precast panes is the cracking and spalling of the concrete at the transverse joint. (Hieber, et al., 2005) This cracking and spalling can allow infiltration of water, salt and chemicals which can negatively affect the longevity of the bridge deck.

The use of full depth concrete panels was the method chosen for the replacement of the Gilford Bridge deck. This method was chosen for many reasons. One of the main reasons is because the use of full depth, full width precast panels does not require the placement of cast-in-place concrete. This replacement method was also chosen because, unlike other methods, the panels are continuous in the transverse direction and do not require the casting of a cold joint at the location of maximum negative moment in the bridge deck. The NHDOT is also collaborating with the University of New Hampshire on new techniques to improve the transverse joint configuration to reduce the possibility of cracking or spalling of the panels. They are also collaborating to develop a procedure to ensure even distribution of the panel load to the girders while maintaining the desired profile of the bridge deck. This procedure will also allow for the setting of the leveling device lengths prior to the placement of the panels, reducing the required assembly time of the bridge deck.

2.2.2. Partial Depth Concrete Panels

Partial depth concrete panels are thin precast panels that act as stay-in-place formwork for a cast-in-place concrete deck. The partial depth panels are typically 0'-3.5" thick and span between the girders of the system. (Chavel, 2012) These panels are placed adjacent to each other along the bridge in a grid pattern. The panels are tied together with reinforcement and concrete is cast on top of the partial depth concrete panels to complete the bridge deck. The overall thickness of the bridge deck using this method is typically 0'-8". (Chavel, 2012) Partial depth precast panels reduce construction time because there is no need to construct or remove temporary formwork.

A common problem with partial depth concrete panel bridge decks is the cracking of the cast-in-place concrete. (Chavel, 2012) This cracking takes place in the cast-in-place portion of the concrete at the transverse joints between the panels and also where the panels bear on the girders. These cracks, even if only partial depth, allow for the infiltration of water and other chemicals which can cause deterioration of the deck elements. Cracking between the panels, and at the location of the girders, is caused by the development of negative moment in these regions. This negative moment induces tension in the top surface of the bridge deck.

This bridge deck replacement method was not a viable option for the replacement of the Gilford Bridge due to the common cracking in the top surface of the bridge deck. Salt and chemical infiltration, due to the anti-icing treatments applied to the roadways in the winter, could affect the longevity of the bridge deck.

2.2.3. NUDECK Panel System

The NUDECK system was designed to be a hybrid bridge deck system that utilized positive aspects of both full depth and partial depth systems. (Hieber, et al., 2005) The NUDECK system consists of full depth precast, prestressed panels. These precast panels span the full width of the bridge and extend between eight and ten feet in the direction longitudinal to traffic flow. The full depth panels consist of "open gaps over girder lines for shear studs and post tensioning; shear stud keys between panels; precast concrete curbs; and sleeves for barrier post attachment." (Fallaha, et al., 2004) The full depth open gap in the concrete, over the girder center lines, is what sets this system apart from other full depth panel systems. Because each panel is divided into a series of separate but connected panels it is possible for this deck system to provide a crown in the roadway while still maintaining a consistent panel depth. (Fallaha, et al., 2004)

The open gap, at the center line of the girders, is for the placement of post tensioning and the welding of shear studs to the top flange of the girder. After the placement and leveling of all of the panels in the system the female by female shear keys are to be grouted and left to cure. After the shear keys are cured, post tensioning cable can be run in the open gaps and stressed. The open channels are then filled with grout and the overlay is placed. (Fallaha, et al., 2004)

One of the key issues with the NUDECK system is the formation of cold joints at the location of the open gaps. The open gaps are located along the center line of the girders at the point of maximum negative moment in the panel. A cold joint is a plane of weakness that occurs when uncured material is cast against cured material. This cold joint is prone to separation, when tension in the joint is experienced, and can allow for the infiltration of water and chemicals through the bridge deck.

The NUDECK system was not a viable option for the replacement of the Gilford Bridge deck due to the concerns about cold joints at the location of maximum negative moment. The NHDOT was very concerned about this joint due to the anti-icing treatments of the roadways in New England. The infiltration of salt and other chemicals could affect the longevity of the bridge deck.

2.3. Accelerated Bridge Construction Projects

Many federal and state agencies are adopting accelerated bridge construction as a viable option for rehabilitation or replacement projects. The Massachusetts Department of Transportation (MassDOT) and the New Hampshire Department of Transportation (NHDOT) are among the many agencies exploring accelerated bridge construction techniques.

2.3.1. MASSDOT FAST-14

The FAST-14 was an accelerated bridge replacement project completed by the MASSDOT on the I-93 Corridor. This project, which took place over the summer of 2011, replaced 14 deteriorated bridges in Medford, MA over the course of 10 weekends.

This extensive accelerated bridge replacement project was put into motion after two large potholes opened up on the northbound side of the I-93 Bridge over Valley Street in Medford. These large potholes were caused by "the decay of concrete and steel attributed to years of postponed maintenance." (Moskowitz, et al., 2010) After more investigation, it was found that fourteen bridges along the length of I-93 in Medford were deteriorating and needed immediate attention.

The Massachusetts Department of Transportation had some very specific goals for the I-93 Bridge deck replacement project. Each one of these goals was developed based on the guidelines set by the FHWA Highways for LIFE program. The goals are as follows. (MASSDOT, 2011)

- Minimize the impact of the project on travelers and communities
- Reduce the construction duration as much as possible
- Use cutting-edge engineering and construction innovation to complete the work
- Communicate important project information and schedule updates with the public in a detailed and ongoing way

With these goals in mind the construction of the first bridge began in June of 2011, but up to this point a great deal of work had already been done. The bridge decks of the Fast-14 project were to be constructed using pre-cast elements. These modular superstructure units were cast off-site well ahead of the project start date in order to avoid delays in the construction process. Constructing the panels off-site allows for a more controlled working environment, eliminating the impact of vehicular traffic, heavy equipment and other occupational hazards. The modular superstructures were also constructed under cover which removes many of the environmental issues that can arise, thus resulting in a better overall product.

The construction process began with the diversion of traffic from the bridge to be replaced to the opposite side of the interstate. This was done by utilizing a moveable barrier system to regulate traffic under normal driving conditions. The position of the moveable barrier was changed to divide the open bridge in half and allow two lanes of traffic to flow in each direction. This inventive traffic pattern allows the construction zone to be free of vehicular hazards while still providing flow of both northbound and southbound traffic. Detours were also put in place to allow for local roads, underneath the bridge, to be closed during the construction period.

Once the traffic was moved to the opposing side of the interstate, work began on preparing the bridge deck for demolition. Excavators and other heavy equipment were used to demolish the superstructure of the bridge. Once the bridge superstructure debris was removed and the abutments and bearing plates prepared, the bridge deck was assembled. The modular superstructure units were placed side by side on the existing substructure and the steel girders of the modular units were connected.

After each superstructure unit was placed, formwork was installed so that concrete could be poured to seal the joint between the adjacent units. This connection is called a closure pour. The concrete used in the closure pour was a very fast setting mix providing a great deal of strength in a short period of time. The use of this fast setting concrete allowed the construction process to continue without much delay.

After the concrete was fully cured, the bridge and roadway was prepared for traffic. Temporary barriers and pavement was installed so vehicular traffic could safely cross the newly constructed bridge. All permanent barriers and other roadwork needs were completed at night over the following months while the bridge remained open for traffic.

The Fast-14 Bridge Replacement Project was made entirely possible by the use of Accelerated Bridge Construction, ABC, techniques. This project was completed in a single construction season. Conventional methods of bridge construction would have required four to five years of constant work to accomplish a similar result. With ingenuity and planning, fourteen bridges were replaced on one of the busiest stretches of roadway in the northeast, with minimal traffic disruption in ten weekends of onsite construction.

This project was completed successfully, but there are some aspects of the modular superstructure units that could lead to problems in the future. The locations of the closure pours, in relation to the girder spacing and traffic pattern, are in the location of maximum positive and negative moments of the bridge deck. This becomes an issue due to the sequence of construction. The closure pour is cast between two precast modular superstructure units creating a cold joint. A cold joint occurs when fresh, uncured concrete is cast against concrete that has already cured. Cold joints are plane of weakness that can lead to cracking and water infiltration through the joint.

Although the use of precast superstructure units greatly reduces the construction time needed for a complete bridge deck replacement it was not considered as an option for the Gilford Bridge deck replacement. This accelerated bridge construction method was not a viable option due to the condition of the girders. The girders of the Gilford Bridge are in excellent condition and are not to be replaced during the deck replacement procedure.

2.3.2. River Street Bridge Boston, MA

During the weekend of April 13, 2012 the MASSDOT replaced the River Street Bridge crossing the CSX and Amtrak lines in Hyde Park Boston. The old bridge had fallen to disrepair due to age, weather, and use. (massDOT, 2012) The MASSDOT decided that the best solution for the bridge replacement would be to implement accelerated bridge construction techniques. Using ABC techniques MASSDOT would greatly reduce the construction time, thus reducing impact and inconvenience to the railroad companies and the surrounding communities.

In order to replace the existing bridge, it was decided that building a full bridge in an adjacent lot then moving it into place would be the best option. The superstructure of the bridge was built on the southwest side of the bridge on shoring towers. For the new bridge, another 18" of clearance was also added so double-decker trains could safely pass beneath the bridge. (Schwartz, 2012) Then on April 13, 2012 the River Street Bridge was closed and crews began to demolish the existing structure. After demolition, crews installed and secured the bearing plates for the new bridge to be set on. The new bridge was moved into place using Self-Propelled Modular Transporters and set down onto the bearing pads. After the bridge was set into place the approaches were prepared and paved.

During this process the bridge was closed to traffic for a total of three days. MASSDOT estimated that if this bridge was rebuilt using conventional methods, instead of ABC techniques, the construction process would have taken over two years to complete. (massDOT, 2012)

The superstructure replacement method used in the River Street Bridge projects is a very popular ABC method if adequate space is available and the entire superstructure of the bridge is in need of replacement. One of the main drawbacks of this bridge replacement method is the impact of environmental effects. Due to the nature of this bridge replacement method, the new bridge is to be constructed adjacent to the existing bridge and not in a controlled environment. Environmental effects during the construction process can diminish the quality of the bridge deck. (Chavel, 2012)

This accelerated bridge construction method was not a viable option for the replacement of the bridge deck of the Gilford Bridge due to the condition of the girders. The girders of the Gilford Bridge are in excellent condition and are not to be replaced in the deck replacement procedure. If the girders were to be replaced, this method of bridge replacement would greatly reduce the closure time of the bridge compared to conventional bridge deck replacement techniques.

2.3.3. NHDOT Gilford Bridge

The Gilford Bridge is located in Gilford, NH and was built in 1963 as part of the Interstate Highway System. The Gilford Bridge carries U.S. Route 3 and NH Route 11 over NH Route 11A. U.S Route 3 and NH Route 11 travel north to south and NH Route 11A travels east to west from Gilford, NH to Laconia, NH. Figure 2 shows the location of the Gilford Bridge in Gilford, NH.

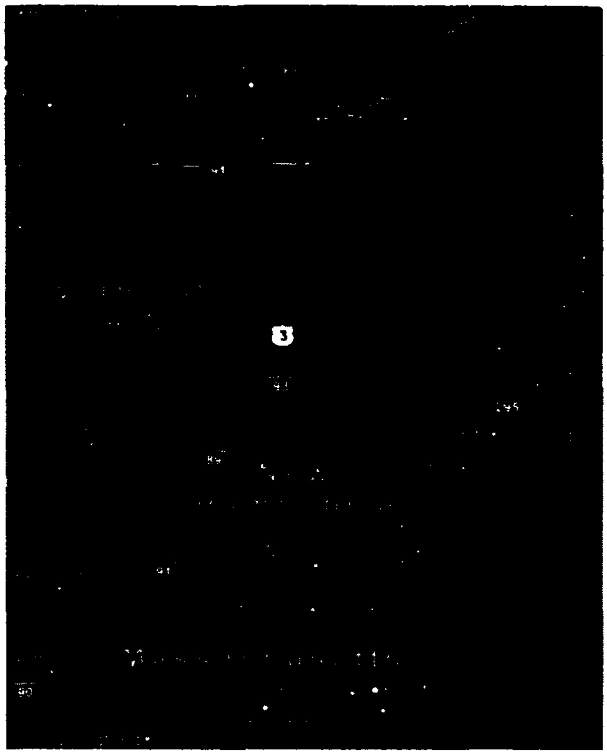

Figure 2: Location of Gilford Bridge

The Gilford Bridge is 76'-0" long, center to center of the abutments, and 49'-10" wide. There are 7 girders supporting the cast-in-place concrete bridge deck at a 23 degree skew to the direction of traffic. The transverse crown of the road is 2% and the longitudinal profile of the road is 1.4%. The bridge has not had any major modifications done in the past except for the addition of protective netting between the girders supporting the bridge deck. The bottom side of the concrete deck is rapidly deteriorating and pieces of concrete are spalling off and have the potential to drop onto the roadway. The protective netting was installed in these problem areas to catch spalling debris and prevent damage or harm to motorists and their vehicles. The bridge deck of the Gilford Bridge is a 0'-7" thick cast-in-place, steel reinforced, concrete deck. The seven girders are WF 36x194 with cover plates located on the middle two-thirds of the bottom flange. Figure 3 shows a side profile of the Gilford Bridge.

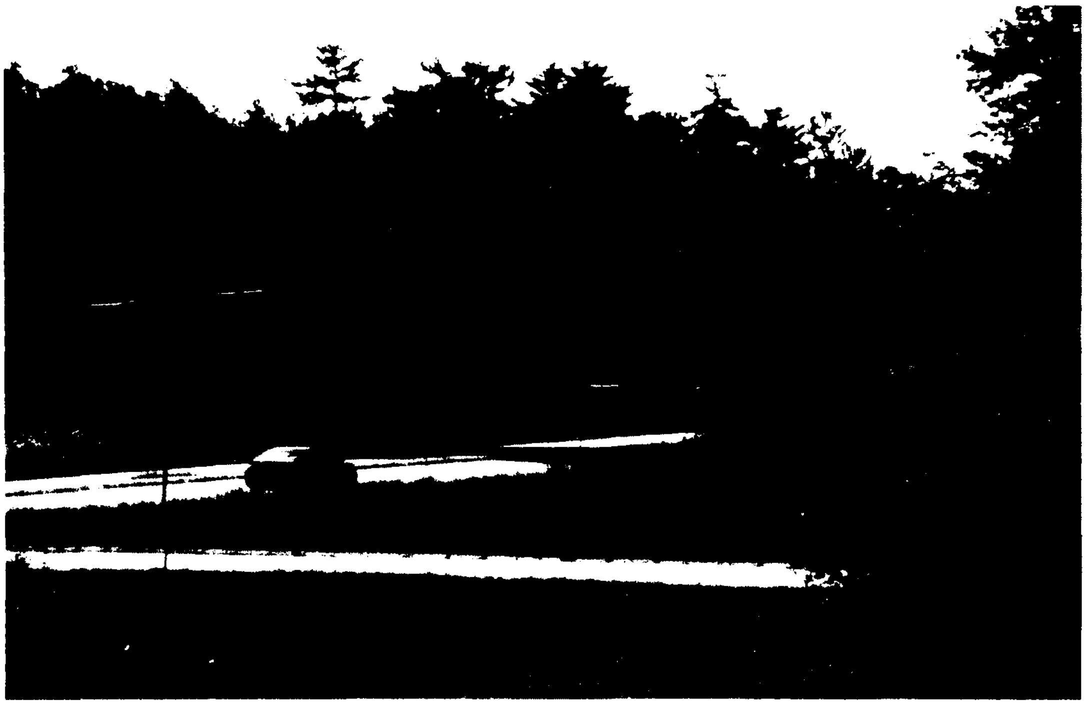

Figure 3: Gilford Bridge Located in Gilford, NH

From the results of field investigations done by the NHDOT, it has been determined that the seven girders, as well as the girder seats and abutments, are in excellent condition and do not need to be replaced. The only maintenance scheduled for the girders is cleaning and repainting with corrosion resistant paint.

The Gilford Bridge deck replacement will be completed using accelerated bridge construction techniques. The technique of choice for this project is the use of full width, full depth precast concrete panels. This method was chosen to avoid the creation of cold joints at the locations of maximum moment in the bridge deck. These precast panels will be prestressed transversely and post tensioned longitudinally, to the direction of traffic, to ensure the entire bridge deck remains in compression.

For the replacement of the existing Gilford Bridge concrete deck, full depth, full bridge width precast panels will be used. Figure 4 displays the precast panel design to be used for the deck replacement of the Gilford Bridge. The full depth panels will be placed transversely to the girders one at a time, starting from the south abutment, to form the new deck. These panels, nine in total, will be transversely prestressed and longitudinally post tensioned in relation to the flow of traffic. This configuration of reinforcement ensures that the bridge deck will remain in compression in all directions once assembled. Both the pre-stressing strand and the post tensioning bars will be located such that the panel is stressed concentrically in relation to the neutral axis of the panel. This will ensure that no camber is induced in the panel due to eccentrically placed reinforcement.

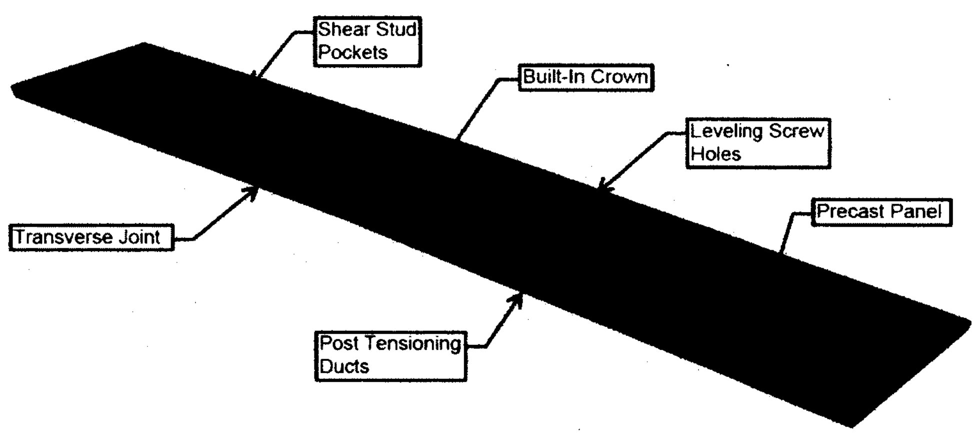

Figure 4: Gilford Bridge Precast Deck Panel

The panels will also be cast with a 2% slope, transverse to the direction of traffic, to form the crown of the road. The crown will be cast into the panel to reduce the amount of asphalt needed to produce the desired wearing surface profile. If the crown was not cast into the panel, the thickness of the asphalt in the center of the panel would be roughly 6" greater than on the edges. This would not only add a great deal of dead weight to the bridge but would also add a great deal of cost.

Each full depth panel will be cast with fourteen leveling screws and twenty-eight shear stud pockets. The fourteen leveling screws, two per girder, will be positioned in line with the girders of the bridge. The leveling screws will be used to accurately position the panel on the girders, at the correct elevation and slope, to create the required final vertical surface profile of the bridge. In the case of the Gilford Bridge, the leveling screws will be set such that the final profile of the bridge deck is at a longitudinal slope of 1.4%.

The twenty-eight shear stud pockets, four per girder, will also be positioned in line with the girders of the bridge and will allow for the welding of eight shear studs per pocket to the top flange of the girder. These shear stud pockets with eight studs each will be grouted to develop composite action between the girders and the bridge deck.

CHAPTER 3

3. LEVELING DEVICE SETTINGS

This research focused on the integration of finite element modeling into the construction phase of accelerated bridge construction. In particular, this research focused on calculating the leveling screw lengths of full depth precast panels so the lengths can be set prior to the placement of the panel on the girders. The common practice for the leveling of precast panels is to place all of the panels onto the girders and then level each panel individually until the desired profile is met. (Chavel, 2012) The process of setting the leveling screw length prior to their placement on the girders would not only simplify the panel placement process but also greatly reduce the time of panel installation.

This research also focused on the procedure and adjustment of the torque of each leveling screw to control the axial load it applies to the girder it is bearing on. Controlling the axial load each leveling screw is applying to the girder is necessary to correctly distribute the dead load of the panel amongst all of the girders and avoid overloading or under loading of the girders.

3.1. Proposed Construction Sequence

The following construction sequence is proposed for the bridge deck replacement of the Gilford Bridge with full depth precast panels. This construction sequence was developed as a culmination of all past accelerated bridge construction research at the University of New Hampshire including the findings from this research. This research focused on the integration of structural analysis software into the construction sequence of full depth precast panels to calculate the leveling device length prior to the placement of the panel. This research also focused on altering the torque applied to each leveling device to control the axial load, due to the dead load of the panel, which each leveling device applied to the girder. The construction sequence used in the lab trials of this research differed slightly from this sequence due to the scope of the research.

After the closure of the bridge to all vehicular traffic the existing concrete deck will be demolished. Once all of the concrete and steel reinforcing is removed from the bridge, the steel girders will be surveyed for camber and prepped for the installation of the precast bridge deck panels. It is imperative for the girders to be clear of any residual steel from the previous shear development system in order to provide a uniform surface for the leveling screws cast in the prefabricated deck panels. Figure 5 shows the girders after demolition of the existing cast-in-place concrete bridge deck.

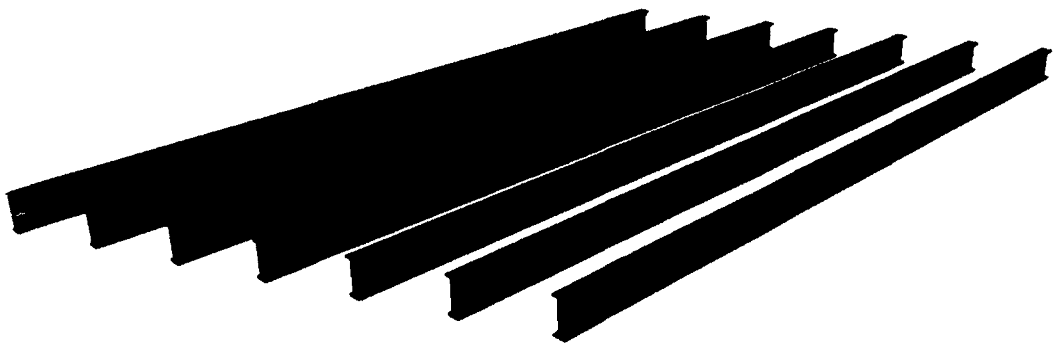

Figure 5: Construction Sequence - Deck Demolished

The girders will be surveyed prior to the placement of the precast panels in order to update the initial vertical profile of the girder system within the structural analysis program. The geometry will be updated within the structural analysis program and the final deflection of the girders under the full load of all precast panels will be calculated.

The calculated deflections of the fully loaded girders will then be used along with the initial survey elevations of all of the girders to calculate the deflected elevations of the girders under the full dead load of the panels. These elevations will then be compared to the desired final profile of the top surface of the completed bridge deck, and a geometric calculation will be completed in order to determine the distance from the top surface of the bridge deck to the top surface of the girder. The thickness of the bridge deck can then be subtracted from this distance, and the length of each leveling screw from the underside of the bridge deck to the top flange of the girder can be calculated. These leveling screw lengths will be set prior to the installation of each precast panel.

After the leveling screw lengths are set for the first precast panel, the panel will be lifted into place and positioned on the girders, starting at one of the abutment. Figure 6 shows the first panel positioned on the girders. Once the first panel is placed in the correct location, and before post tensioning, the leveling screws will be checked twice for torque in a diagonal cross pattern. The torque of each leveling screw is used to control the axial force each screw applies to a girder due to the dead load of the panel. Calculations must be done prior to the installation of each panel to determine the necessary leveling screw torque setting to ensure the correct distribution of panel dead load is applied to each girder. This calculation will be governed by the effective tributary panel area supported by each leveling screw.

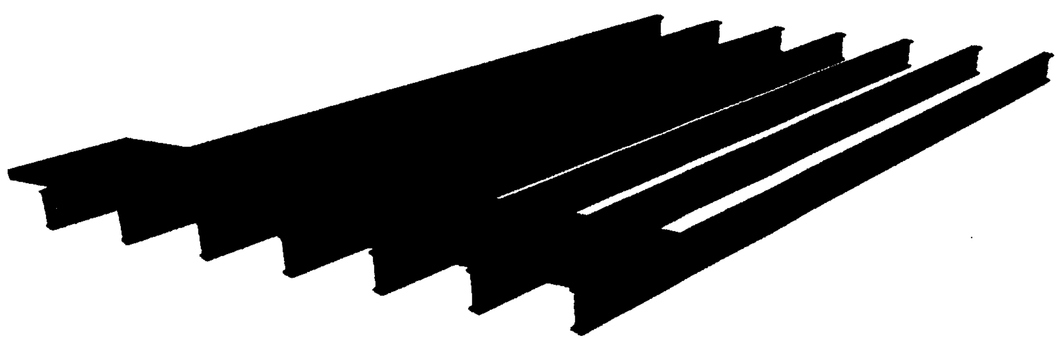

Figure 6: Construction Sequence - First Panel Placed

After the torque of the leveling screws has been adjusted, the panel will be post tensioned in the longitudinal direction of the bridge, inducing compression in the concrete. The post tensioning bars of each panel must be stressed prior to the installation of the next panel to ensure adequate spacing for the threaded bar coupling within the post tensioning block outs. Subsequent panels will be post tensioned to installed panels. This will be accomplished by coupling the post tensioning bars of the newly installed panel to the post tensioning bars of the previous panel, creating a continuous longitudinal post tensioned deck. The post tensioning of the panels will utilize a threaded bar system. Conical shaped nuts threaded to the steel bar and bearing plates cast into the panel will be used at each end to retain the tension applied to the bar. The precast panel will be post tensioned to ensure the concrete remains in compression.

After the leveling screws have been checked for torque and the panel is post tensioned, another survey of the girders is done. This survey is to determine the deflection of each girder due to the application of the dead load of the installed panels. The geometry of the deflected girders will be compared to the girder deflections determined by the structural analysis program. This comparison will show any difference between the deflections calculated in the structural analysis software and the actual deflections of the girders.

Once the survey is complete and the deflections of each girder deemed satisfactory, the leveling screws of the second precast panel can be set to their required length and the panel can be lifted into place. The panel will be positioned on the girders leaving roughly a 0'-6" gap between itself and the preceding panel. Figure 7 shows the placement of the second panel leaving a gap between panel one and panel two. A gap is left between the two panels in order to couple the post tensioning bars together as well as apply a structural sealant to both sides of the transverse panel joint. The torque of the leveling screws will then be checked to ensure uniform loading of the girders. A post tensioning bar will be placed through the second panel and joined to the already stressed post tensioning bar of the first panel with a threaded coupling.

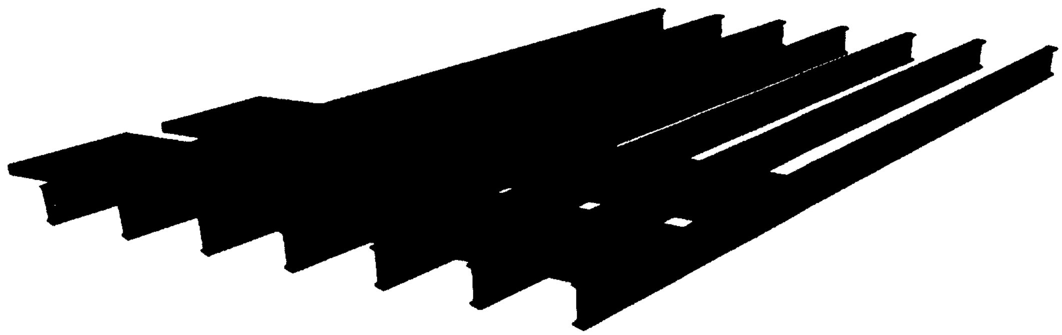

Figure 7: Construction Sequence - Second Panel Placed with Gap

Before the panels are post tensioned together a structural adhesive is applied to the edge of each panel to form the transverse joint. This structural adhesive is applied to a thickness of half the longitudinal tolerance of the panel. This material not only seals the joint but also provides the panels with a uniform bearing surface at the transverse joint during post tensioning and transfers shear under live load. If this sealant was not applied prior to post tensioning, the panels would develop non-uniform stresses within the transverse joint due to high and low spots within the joint. This non-uniform stress, if high enough, could cause the concrete to crack and spall at the point of contact of the two panels.

After the structural joint sealant is applied to both sides of the transverse joint, the panels can be joined together until some of the sealant squeezes out of the top and bottom, ensuring full coverage within the joint. After the panels are joined, the torque of the leveling screws of the panel will be checked again to ensure equal distribution of the dead load of the panel still exists. The panels are then left to sit until the sealant has cured enough to support the compressive load applied during post tensioning.

After the structural joint sealant has cured, the panels can be fully post tensioned together by pulling from the outside edge of the second panel through both panels to the beginning end of the first panel. Figure 8 shows the two panels post tensioned together. Because the two post tensioning bars are coupled together, they act as a single bar when stressed. This staged post tensioning process ensures uniform compressive stresses in the panels.

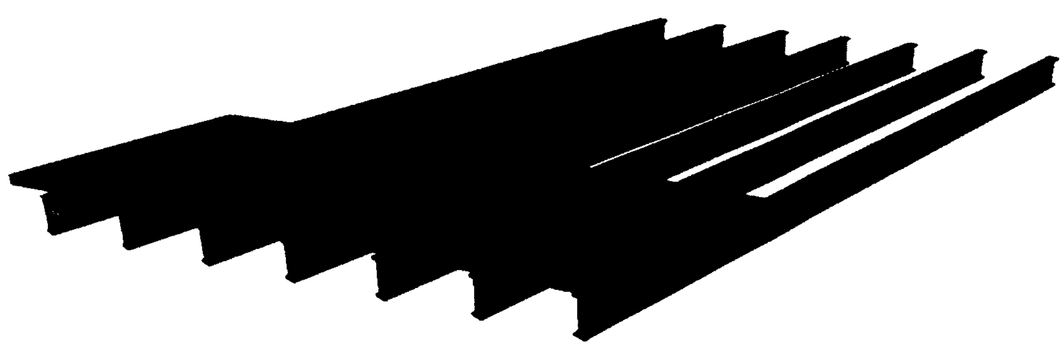

Figure 8: Construction Sequence - Second Panel Post Tensioned

While the panels are being post tensioned together, the girders will once again be surveyed and checked against the calculated deflections of the structural analysis model. Any corrections in the loading of the girders can once again be determined for the placement if the next panel. This research did not address the process of correcting for differences in deflection between the calculated and measured deflections of the girders.

This process of setting leveling screws, placing panel, adjusting the leveling screw torque, installing post tensioning bars, applying structural sealant to the transverse joint, joining panels together, rechecking the leveling screw torque, allowing polymer to fully cure, surveying the girders, and fully post tensioning the panels together will be repeated for each precast panel that is to be installed. Figure 9 shows all of the panels placed onto the girders and post tensioned together.

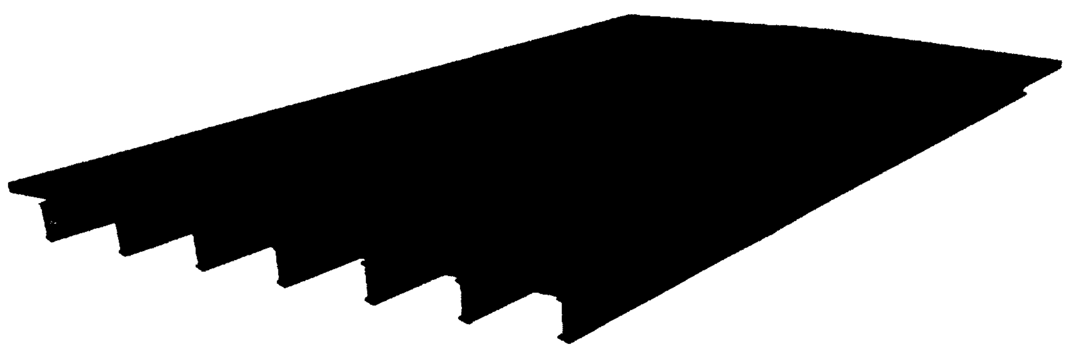

Figure 9: Construction Sequence - Full Deck

After the installation of the precast panels, shear studs are welded to the top flanges of the girders within the shear stud pockets of the precast panels. Forming will also be applied to the underside of the panels for the pouring of the haunch. Once the shear studs are welded and the forming in place the haunch and shear stud pockets can be grouted. After the haunch has cured the leveling screws can be removed and those holes grouted.

The post tensioning ducts must also be fully sealed and grouted to prevent corrosion of the threaded bars. This has been done in the past using a methyl methacrylate injection procedure. A vacuum is induced at one end of the post tensioning duct and methyl methacrylate is injected into the other end. Once the duct is filled, the methyl methacrylate is allowed to cure, fully encapsulating the post tensioning bars.

Following the installation of the precast concrete panels, an asphalt membrane will be applied to the concrete deck surface. This asphalt membrane seals the bridge deck as well as provides a good contact surface for asphalt pavement. The bridge deck and approaches can then be paved, guard rails installed, and the bridge can be opened to traffic.

3.2. Experimental Setup

For the testing of the accelerated bridge deck replacement method, a four girder lab model was built at the University of New Hampshire's structures laboratory in Kingsbury Hall. Figure 10 shows the four girder lab model with all of the precast panels loaded.

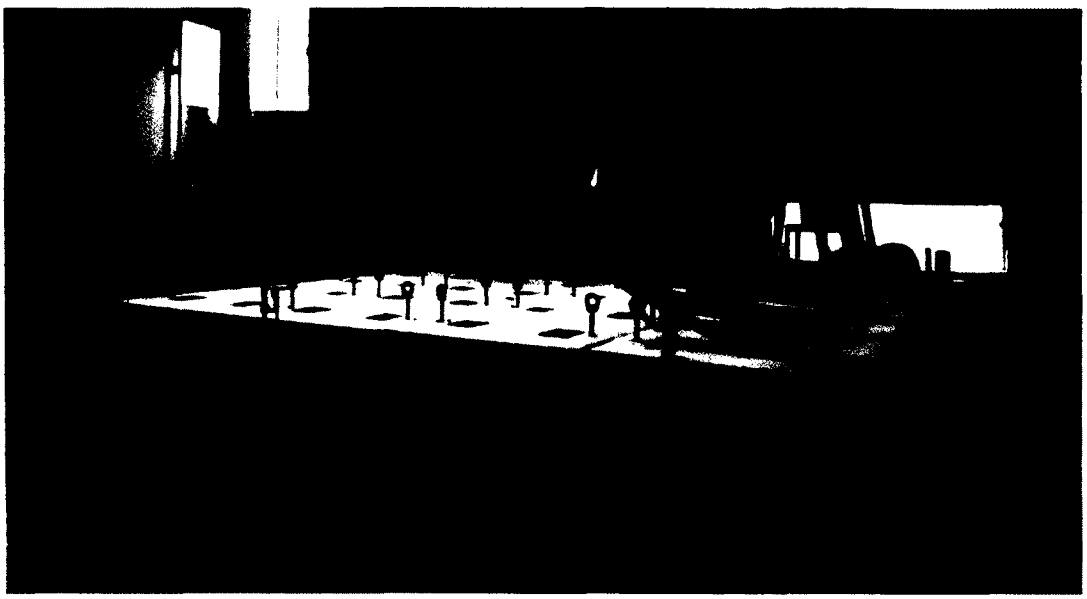

Figure 10: Four Girder Lab Model Fully Loaded

This lab model was built to match some of the characteristics of the Gilford Bridge. These characteristic included the skew of the girders, panel configuration and panel loading process. This was done to identify any potential problems in the proposed construction sequence.

3.2.1. Girder Configuration

This bridge model consisted of four, twenty-five foot long, W8x24 girders placed at a twenty-three degree skew. These girders were connected with six, C6x10.5, channel sections which acted as diaphragms to protect against lateral torsional buckling of the girders. The girders and diaphragms were connected with welded gusset plates and grade-8 hardware. Additional wood blocking was placed between the girders at five foot intervals to provide more torsional stability.

The end of each girder was designated as either the I-End or the J-End. The I-End of the girder was the Southern end and the J-End was the Northern end. Figure 11 displays the I-End of the four girder lab setup looking down the girders towards the J-End (Southern end looking North).

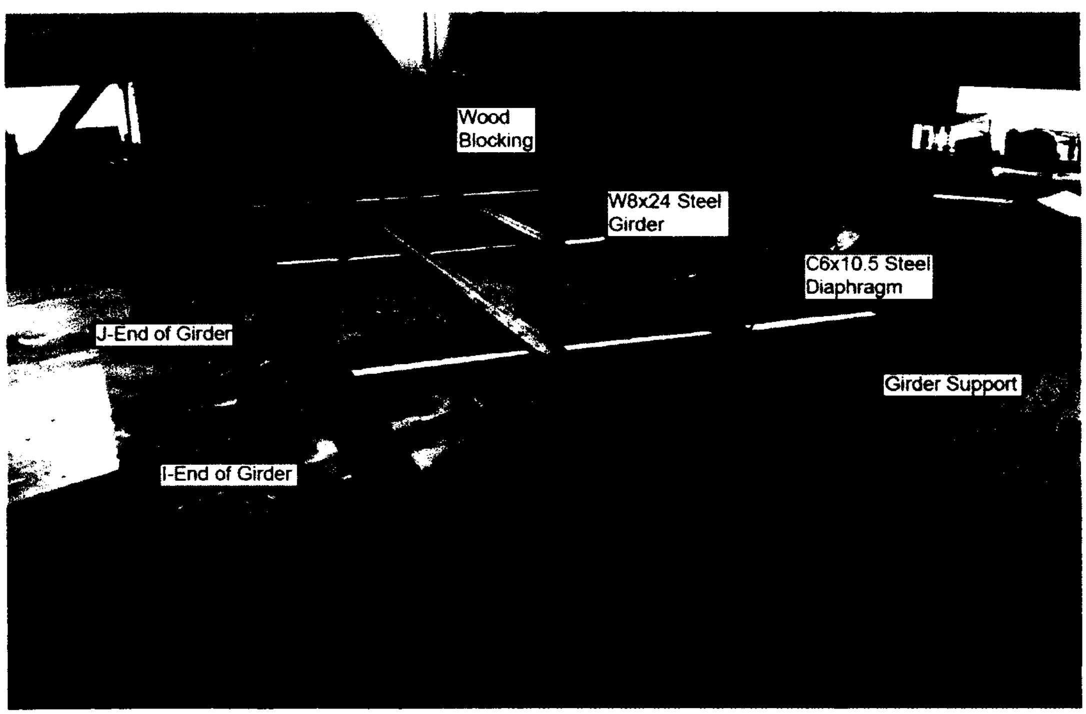

Figure 11: Four Girder Lab Model

The four girders were set on supports consisting of rounded bearing plates cast into a concrete block as seen in Figure 12.

Figure 12: Bearing Plate Cast in Concrete Block

This type of support acted as a roller allowing for translation horizontally while vertically supporting the structure.

3.2.2. Girder Survey Procedure

The top flanges of the girders were surveyed throughout the assembly of the concrete bridge deck. An initial survey was taken to determine the geometry of the unloaded girders. The girders were also surveyed after each panel was placed in position and leveling screws torqued to determine the deflection due to loading. It is important to survey the girders after each panel is placed to evaluate the difference between actual girder deflection and model deflections.

Due to the placement of the panels on the four girders, it was only possible to survey the top flange of the girders by placing the survey rod through the shear stud pockets when panels were loaded. This is because the shear stud pockets were the only clear opening to the top of the girder when panels are installed. During a full scale construction process it would be beneficial to survey the bottom flange of the girders, from the underside of the bridge, and adjust the elevation readings by the depth of the girders.

Survey points were chosen at four locations, within the shear stud pockets of the panels, along each girder as well as the locations of the supports. The six locations were 0'-6", 6'-8", 11'-0", 13'-2", 17'-6", and 24'-6" from the I-End of each girder. Figure 13 shows the location of each survey point and its distance from the I-End of the girder.

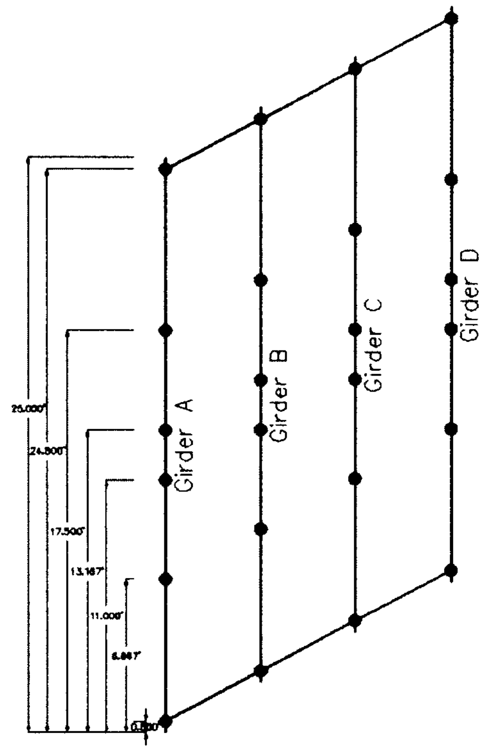

Figure 13: Girder Survey Locations

These six locations on each girder were used throughout the laboratory tests as well as in the structural analysis model to determine initial girder geometry for deflection calculations.

3.2.3. Panel Configuration

The bridge deck of the lab model consisted of four precast reinforced concrete panels. Figure 14 shows the four precast panels. These panels were sixteen feet wide in the transverse direction of the girders and four feet long in the longitudinal direction of the girders. The panel ends were also cast with a twenty-three degree skew, matching the skew of the girder system.

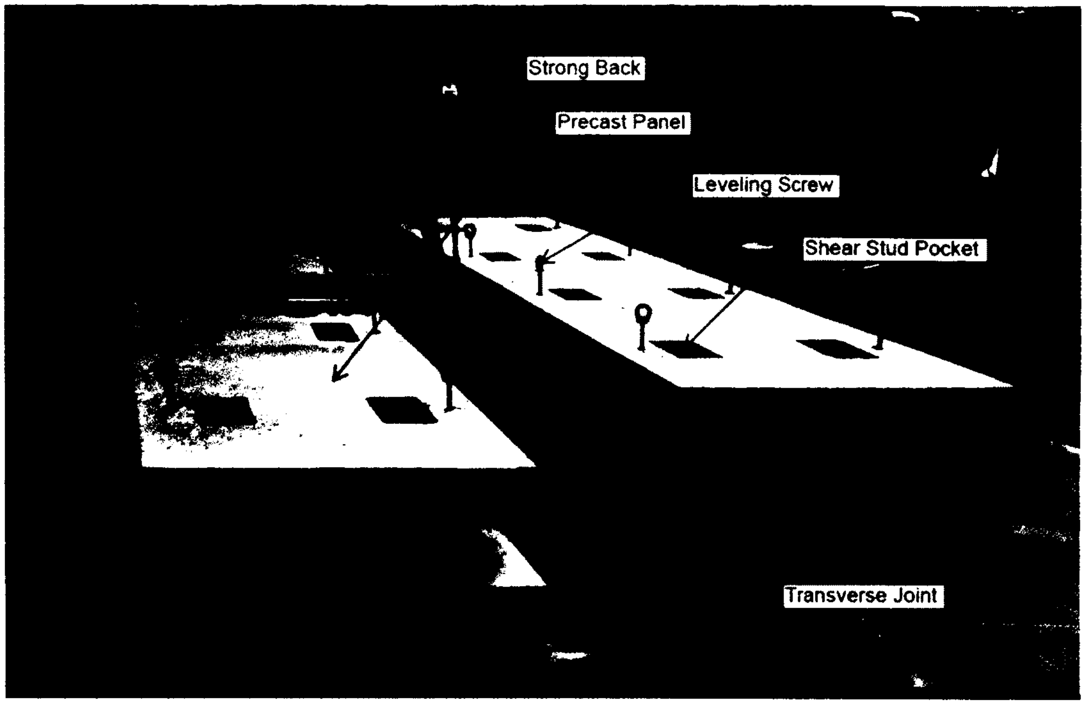

Figure 14: Precast Panels

Each of the four precast panels was cast with eight leveling screws and eight shear stud pockets. The eight leveling screws, two per girder, were aligned longitudinally to the girder. These leveling screws protrude from both the top and the bottom of the concrete panel. This allows for the leveling screws to also be used as lifting points. A female threaded eye hook was attached to the top side of the eight leveling screws which was then attached to hooks and lifted using a strong back. Figure 15 shows the precast panel being lifted with the strong back.

Figure 15: Strong-back Lifting Panel at Eight Locations

The eight shear stud pockets, two per girder, were also aligned longitudinally to the girders. Figure 16 shows the eight shear stud pockets cast into the panel. These pockets allowed for access to the top flange of the girders for welding on shear studs. These shear stud pockets would then be grouted with a high strength concrete grout to join the shear studs to the bridge deck. The shear studs provide a means to transfer horizontal shear, created by vehicles moving along the bridge, from the bridge deck to the girder system and ultimately to the abutments. This integration of components allows for composite action throughout the bridge system.

Figure 16: Shear Stud Pockets

This research did not include the welding of shear studs to the girders or the grouting of the shear stud pockets.

Each precast panel was cast with six post tensioning ducts located at the neutral axis of the panel running longitudinal to the girders. Figure 17 shows the post tensioning duct and block out. Post tensioning are inserted in the ducts and connected to the respective bar in the adjacent panel. These bars are then stressed, inducing compression into the concrete as well as joining the bridge panels together.

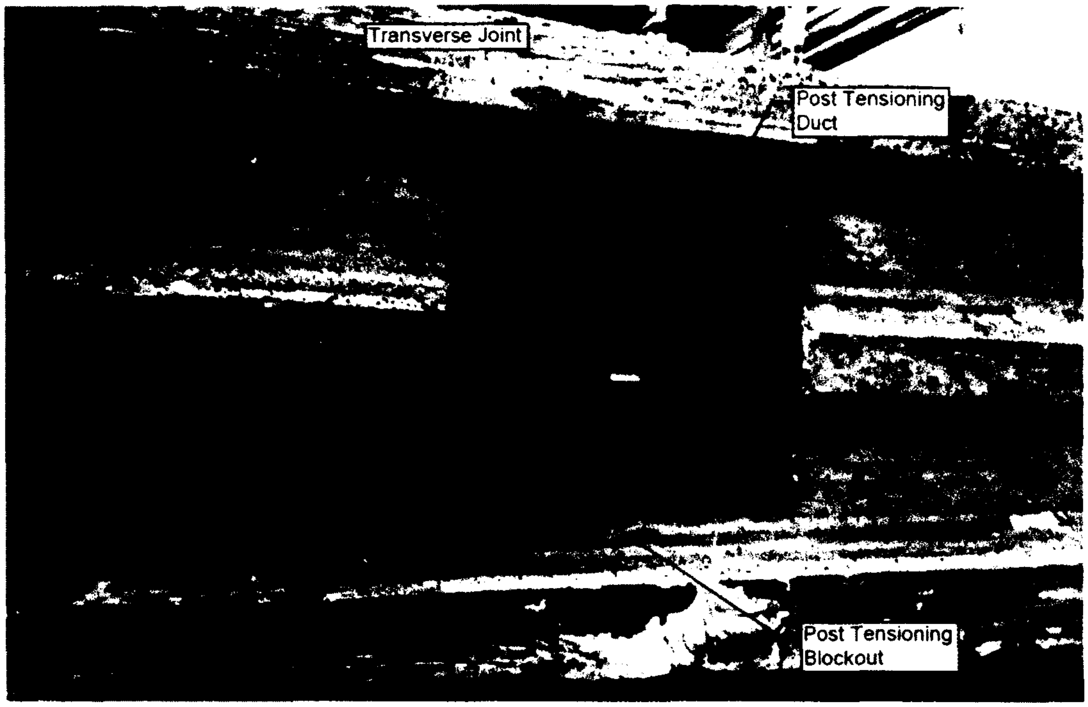

Figure 17: Post Tensioning Duct and Block Out

The use of post tensioning bars, to join the panels together, was not a part of this research.

The transverse joints of the panels were cast with a double tongue and groove. It has been confirmed (See Section 2.1.2) that this tongue and groove joint allows for the transfer of vertical shear between bridge deck panels.

The four precast panels were loaded onto the girders concentric to the midpoint of the girders. This loading placement caused the maximum deflection in the girders.

3.2.4. Precast Panel Construction

During the construction, plywood forms were used to cast the concrete deck panels. Insufficient cross bracing was installed on the forms resulting in warping along the transverse joints of the panels. Figure 18 shows the warping of the panels along the transverse joint. The warping was greater than what would be accepted as within the standard width tolerance. It is not expected that the warping of the panels affected the results of the trials of this research. The only noticeable affect the warping of the transverse joint would have would be during the post tensioning process. The post tensioning of the panels was not in the scope of this research.

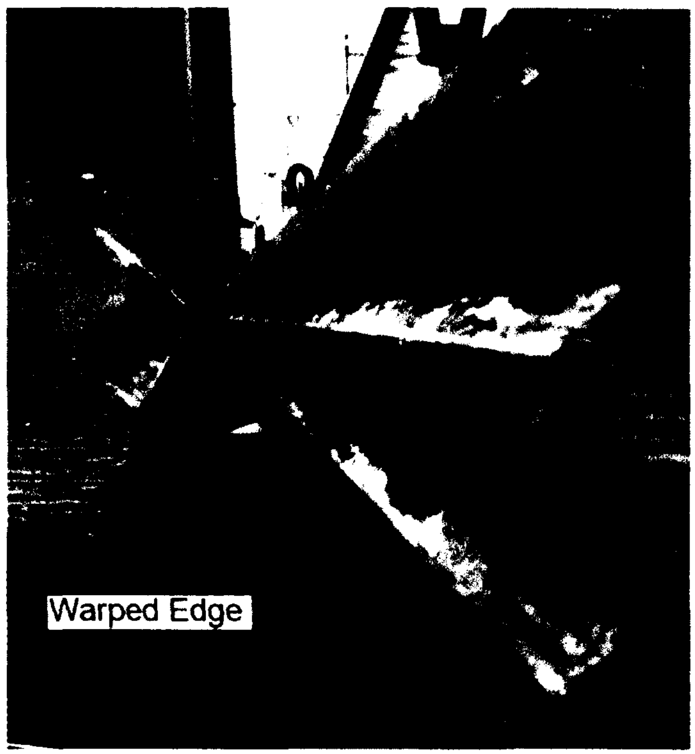

Figure 18: Warped Concrete Bridge Deck Panel

The concrete bridge deck panels were constructed on the floor of the structures laboratory in Kingsbury Hall. It was observed after stripping the panels from the molds that the thickness of the deck panels varied as much as 0'-0.25". The panels were constructed using Self Consolidating Concrete (SCC) to facilitate sufficient flow to eliminate vibrating and screeding to achieve a level surface. It was found, after the concrete panels had cured and been removed from the forms, that the floor of the structures lab was not consistently level or flat. This caused the panels to have an uneven underside. Figure 19 shows the uneven underside of the precast panel.

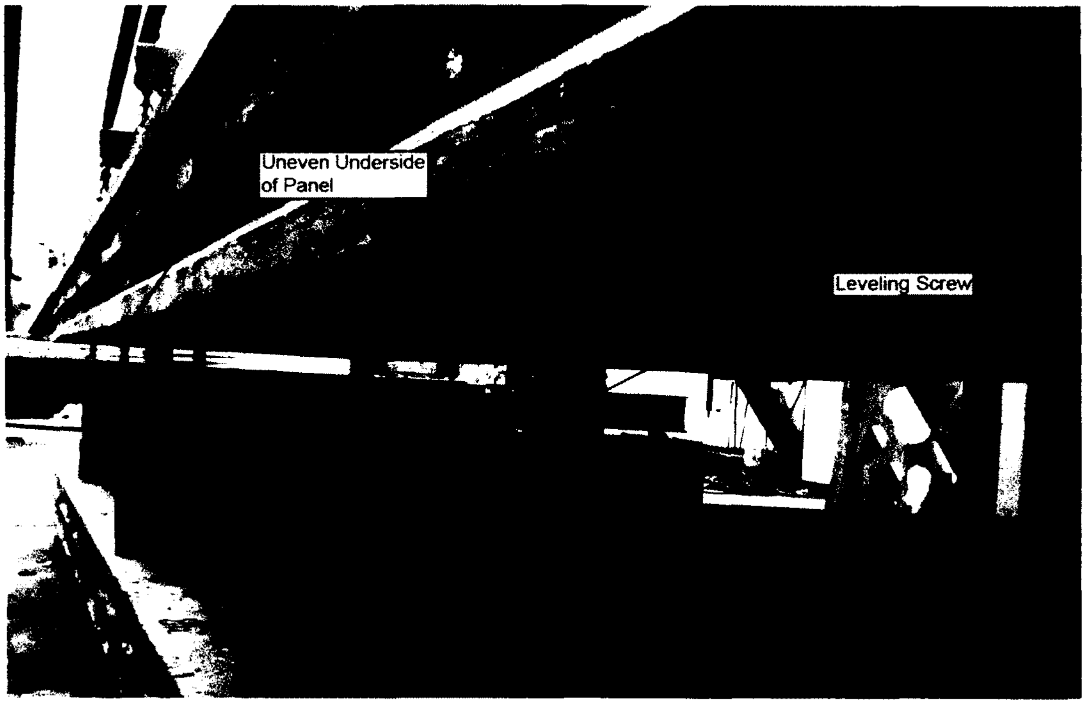

Figure 19: Underside of Precast Panel

This inconsistent profile was found to affect the ability to accurately set the leveling screw lengths beneath the panel. This was because the leveling screw lengths were calculated from the bottom of the panel elevation with the assumption that the bottom surface of the panel was flat. This discrepancy was not accurately corrected for within the lab trials outlined in this research and its affects should be addressed before additional research is performed with these panels.

3.2.5. Precast Bridge Deck Panel Weight

In order to calculate girder deflections during loading in the computer model, the magnitude and position of the leveling screw load must be known. In this case the magnitude being applied to the girders is the weight of each bridge deck panel divided by the number of leveling screws in the panel. The position of each load is at each of the leveling screw locations. The weight of each panel can be estimated by calculating the volume and multiplying it by its specific weight. It was determined, after two complete lab trials that the weight of each panel needed to be known to a greater accuracy than estimation could provide. This was determined because for the first two lab trials the structural analysis model was overestimating the deflections of the girders compared to the lab deflections.

As a part of this research the precast panels were weighed to determine their true dead load. To determine the weight of each panel, the New Hampshire State Police brought six HAENNI Wheel Load Scales to the structures laboratory to physically weigh each precast panel. The State Police brought a total of six scales with them to weigh the panels. Six scales were used in order to distribute the weight of the panel over multiple scales and ensure that the panel did not crack under its own weight due to a large unsupported clear span.

Once the entire weight of each panel was supported by the wheel load scales, the readings were tabulated and summed. Table 1shows the summary of the scale readings and the resulting panel weights.

Table 1: Measured Panel Weight

Measured Panel Weight (lb.)							
Panel Number	Scale 1	Scale 2	Scale 3	Scale 4	Scale 5	Scale 6	Total
Panel 1	1000	1800	0	0	3300	0	6100
Panel 2	2900	0	0	2250	450	200	5800
Panel 3	1000	950	1100	0	2350	350	5750
Panel 4	850	0	4000	1000	0	200	6050

The average weight of the four panels was 5925 lb. This average weight was much less than the calculated weight of 6720 lb. and could account for some of the error between girder deflections. This average panel weight was used throughout this research as the dead load of a single panel.

3.2.6. Leveling Screw Torque

As a part of this research a study was conducted in the Structures Laboratory of Kingsbury Hall on the relationship between the torque that is applied to a leveling screw and the axial load transfers to the girder. This study was performed in order to determine and control the amount of axial load each leveling screw transfers to a girder such that the dead load of the panel can be accurately distributed to all of the girders.

When a panel is set into place on the girders, the leveling screws bear directly on the top flange of the girder. The entire weight of the panel is supported by the leveling screws, which applies the panel weight as point loads to the girders. In order to equally distribute the weight of the panel to each of the girders, the axial load that each leveling screw applies to the girder must be controlled.

This study was conducted using a load cell attached to a Data Acquisition (DAQ) system. This system was controlled by a program written using the National Instrument's

LabVIEW software package. (NI Corp., 2013) The leveling screws located within the precast panels in the Structures Laboratory were used, one at a time, to apply an axial load to the load cell. Figure 20 shows one of the leveling screws of a precast panel bearing on the load cell. The weight of the panel was initially supported by all of the leveling screws except for the one that was to bear on the load cell. This was done to ensure that no initial load was being transferred to the load cell due to the weight of the panel. A steel shim was placed between the leveling screw and the bearing surface of the load cell in order to replicate the friction surface between the leveling screw and the top flange of the steel girder.

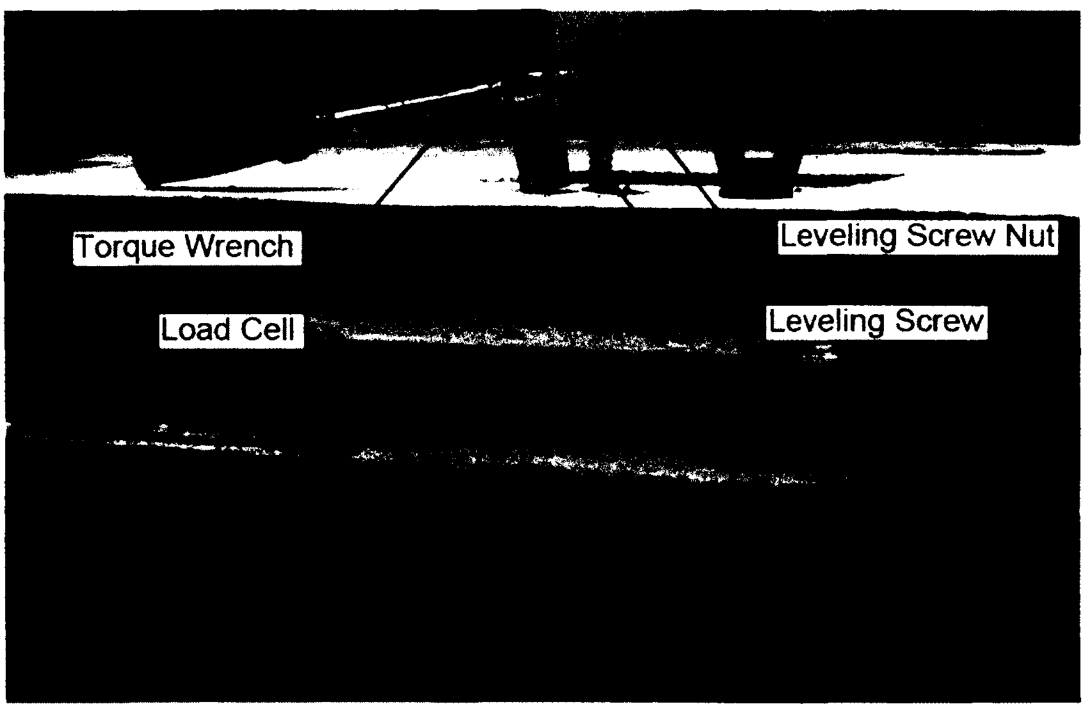

Figure 20: Leveling Screw in Precast Panel

The leveling screw was turned clockwise with a torque wrench incrementally to apply an axial load to the load cell. Figure 21 shows the leveling screw bearing on the steel shim to apply an axial load to the load cell. The leveling screw was turned until a predetermined torque was reached at which point the axial load being applied to the load cell was recorded. The torque was increased in increments of fifteen inch-pounds from twenty-five to one-hundred and ninety inch-pounds. After each increase in torque, the axial load applied to the load cell was documented.

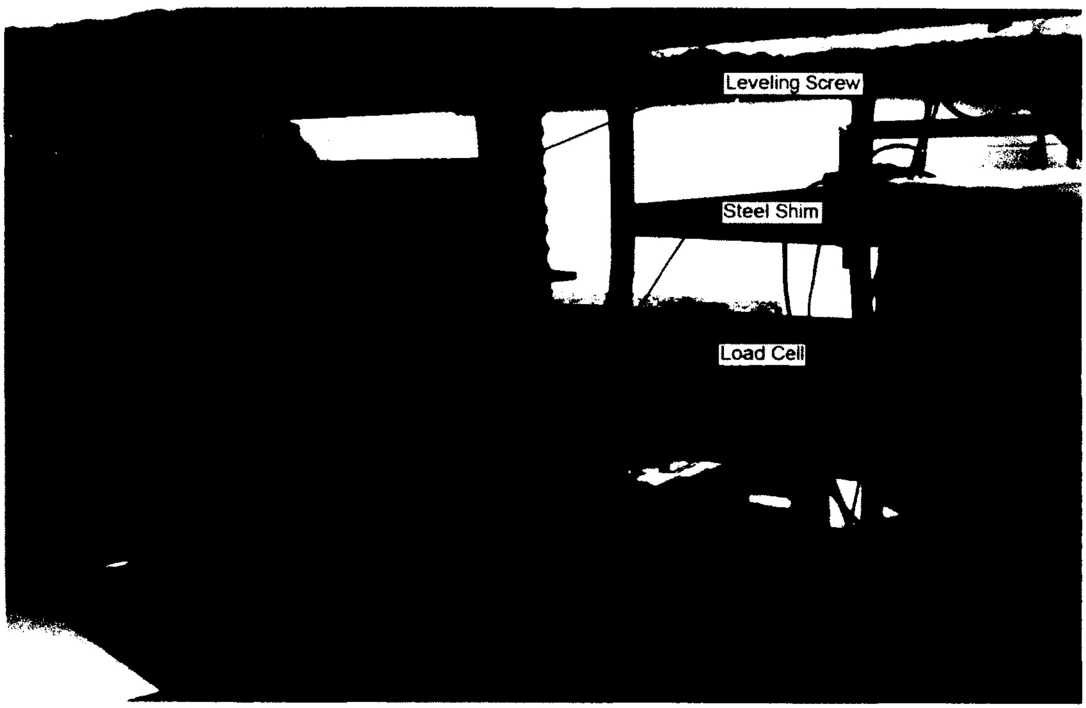

Figure 21: Leveling Screw and Load Cell

This process was repeated four times with four different leveling screws. The axial load results for each torque setting were averaged and the standard deviation of the results was calculated. The averaged results of this study are displayed in Table 2.

Table 2: Leveling Screw Torque vs. Axial Load

Leveling Screw Torque vs. Axial Load Test			
Average Results		Statistics	
Torque (in-lb.)	Axial Load (lb.)	Std Dev	Std Dev/Avg
25	162.50	12.58	7.74%
40	263.75	18.87	7.16%
55	375.00	21.21	5.66%
70	491.25	28.39	5.78%
85	605.00	40.41	6.68%
100	732.50	38.84	5.30%
115	826.25	51.70	6.26%
130	945.00	30.82	3.26%
145	1028.75	27.80	2.70%
160	1181.25	51.70	4.38%
175	1281.25	60.88	4.75%
190	1355.00	58.74	4.33%

The average axial loads were also graphed against the torque values in order to determine an equation for the relationship. The relationship was modeled with a linear trend line. The equation of the trend line was not modeled with a y-intercept of zero because it was assumed that a measurable amount of friction loss existed between the leveling screw and the supporting metal and concrete. This friction loss existed regardless of the amount of grease located at the interface of the leveling screw and these two materials. The y-intercept of this equation, roughly 3.4 in.-lbs., represented the friction losses within the interaction. This is not a large amount as the leveling screws turned freely by hand when no load was applied to them.

Figure 22 illustrates the linear relationship between the torque applied to the leveling screw and the axial force it exerts on the surface below. This figure also shows the linear trend line along with the equation of this relationship. This equation was used in determining the torque applied to each leveling screw during the panel installation process.

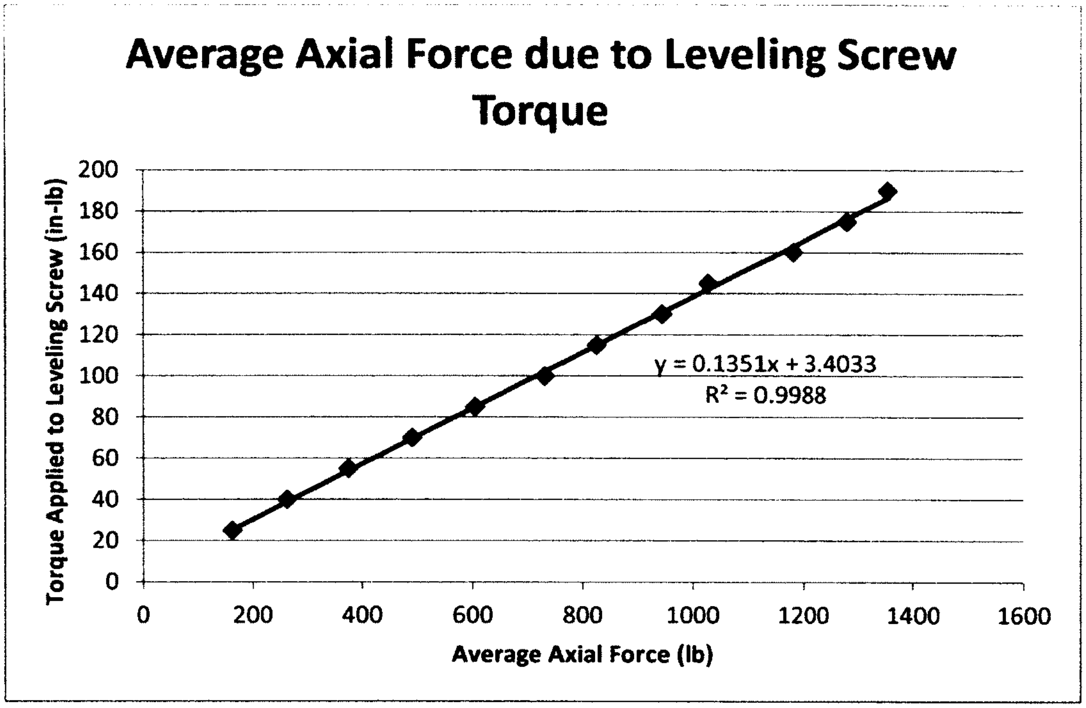

Figure 22: Average Axial Force vs. Leveling Screw Torque

The equation for calculating the torque needed to achieve a given axial force is:

$$t = 0.1351P + 3.4033$$

$$t = torque\ applied\ to\ leveling\ screw\ (in.-lb.)$$

$$P = axial\ load\ of\ leveling\ screw\ (lb.)$$

The average weight of each precast panel is 5925 pounds.

The leveling screw torque calculation is as follows:

Weight of Precast Panel (W)

$$W = 5925\ lb.$$

Axial Load per Leveling Screw (P)

$$P = \frac{W}{n}$$

$$(Due\ to\ each\ leveling\ screw\ supporting\ the\ same\ tributary\ area)$$

$$n = 8\ leveling\ screws\ per\ panel$$

$$P = \frac{5925\ lb.}{8}$$

$$P = 740.625\ lb. per\ leveling\ screw$$

Leveling Screw Torque (t)

$$t = 0.1351P + 3.4033$$

$$t = 0.1351(740.625\ lb.) + 3.4033$$

$$t = 103.46\ in. -lb.$$

According to this study, each of the eight leveling screws will be torqued to roughly 103.5 in.-lbs. in order to equally distribute the precast panel's weight to the girders.

3.3. SAP2000® Model

The four girders and the six diaphragms of the lab model were modeled in the structural analysis program SAP2000® using frame elements. SAP2000® frame elements are straight lines between two joints which are "used to model beams, columns, braces, and truss elements in planar and 3D systems." (CSI Berkeley, 2012) These elements account for biaxial bending, torsion, axial deformation and biaxial shear within the element during analysis. (CSI Berkeley, 2012) Frame elements were used in order to reduce the complexity of the analysis and to allow for faster and simpler model updating. The SAP2000® model was built to the dimension and profile of the four girder lab model. The geometry of the SAP2000® model was based on measurements taken of the lab model to the nearest 1/16th of an inch. The elevations of the lab model used in the SAP2000® model were determined through a survey to the nearest 1/32nd of an inch. Figure 23 shows a plan view of the structural analysis model with dimensions.

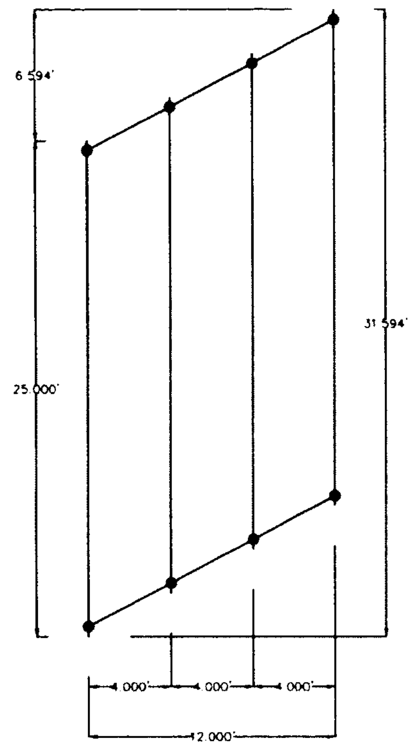

Figure 23: Structural Analysis Model – Plan View with Dimensions

The dimensions shown in the plan view of the SAP200® model remained the same throughout this research. The only alteration made to the lab model throughout this research was the updating of the girder elevations. This was necessary because for each lab trial the support elevations were altered. This was done to provide more variability between trials.

3.3.1. Material Properties

Before a model can be built, the properties of the materials that will be used must be defined. In the case of the four girder lab model all of the elements used, the girders and diaphragms, were steel members. The material properties for the steel members were defined in SAP2000® as A36 steel. Table 3 shows the properties of A36 steel used in the SAP2000® analysis

Table 3: A36 Steel Material Properties

A36 Steel Material Property Data			
Characteristic	Symbol	Value	Units
Weight per Unit Volume	γ	0	lb./ft^3
Modulus of Elasticity	E	29000000	psi
Poisson's Ratio	U	0.3	-
Coefficient of Thermal Expansion	A	6.50E-06	1/°F
Shear Modulus	G	11153846	psi
Minimum Yield Stress	F_y	36000	psi
Minimum Tensile Stress	F_u	58000	psi
Effective Yield Stress	F_{ye}	54000	psi
Effective Tensile Stress	F_{ue}	63800	psi

The weight per unit volume of the steel was set to zero pounds per cubic foot. This was done in order to eliminate additional deflection in the analysis due to the dead load of the girders. The geometry of the four girder lab model was based on the surveyed existing conditions of the girder. These existing conditions already accounted for the deflection of the girders due to their own dead load.

3.3.2. Section Properties

Along with the material properties, the section properties of the materials to be used in the model must be defined. The section properties defined in SAP2000® for the girders and the diaphragms are displayed in Table 4.

Table 4: Frame Element Section Properties

Frame Element Section Properties				
Characteristic	Symbol	W8x24	C6x10.5	Units
Outside Height	t3	7.930	6.000	in.
Top Flange Width	t2	6.500	2.030	in.
Top Flange Thickness	tf	0.400	0.343	in.
Web Thickness	tw	0.245	0.314	in.
Bottom Flange Width	t2b	6.500	2.030	in.
Bottom Flange Thickness	tfb	0.400	0.343	in.
Cross-Sectional (axial) area	A	7.080	3.070	in.^{2}
Torsional Constant	J	0.346	0.128	in.^{4}
Moment of Inertia about 3 Axis	I	82.700	15.100	in.^{4}
Moment of Inertia about 2 Axis	I	18.300	0.860	in.^{4}
Shear Area in 2 Direction	A_2	1.943	1.884	in.^{2}
Shear Area in 3 Direction	A_3	4.333	1.393	in.^{2}
Section Modulus about 3 Axis	S	20.858	5.033	in.^{3}
Section Modulus about 2 Axis	S	5.631	0.562	in.^{3}
Plastic Modulus about 3 Axis	Z	23.100	6.180	in.^{3}
Plastic Modulus about 2 Axis	Z	8.570	1.140	in.^{3}
Radius of Gyration about 3 Axis	r	3.418	2.218	in.
Radius of Gyration about 2 Axis	r	1.608	0.529	in.

The girders of the lab model were W8x24 wide flange sections and the diaphragms were C6x10.5 channel sections.

3.3.3. Draw Objects

After the material and section properties were defined, objects were drawn. Special joints were drawn at the locations of the girder ends as well as girder support locations and diaphragm connections. Joints were also defined at survey and leveling screw locations. Figure 24 shows all of the joints drawn for the structural analysis model. In SAP2000®, joints are defined as a point of connection between structural members.

(CSI Berkeley, 2012) "Constraint conditions are also applied to joints to establish correlation among their displacement (degrees of freedom)." (CSI Berkeley, 2012)

Figure 24: Structural Analysis Model Joints

After the joints were drawn, 3D frame elements were drawn to connect the joints. The 3D frame elements were drawn to match the four girder lab model configuration. Figure 25 shows the frame element girders and diaphragms connecting the joints in the system.

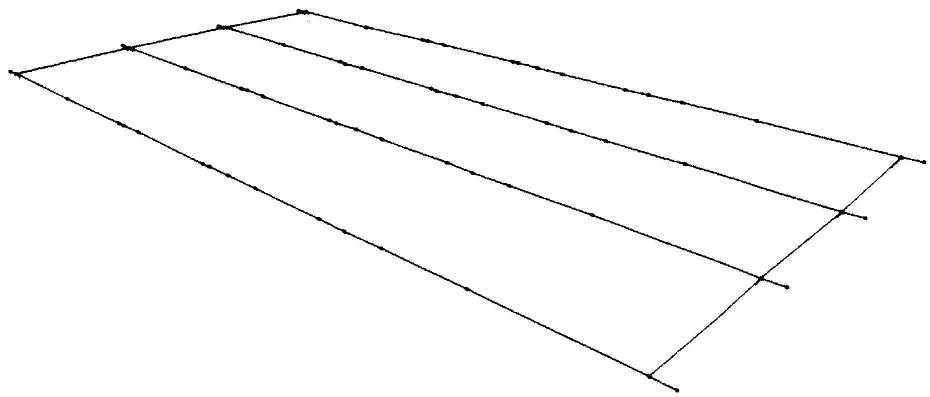

Figure 25: Special Joints and Frames Drawn

3.3.4. Support Conditions

The frame elements of the SAP2000® model were supported 0'-6" feet from each end. The I-end support was defined as a pinned connection restricting translation in all directions and allowing rotation in all directions. The J-end support was defined as a modified pin connection restricting translation in the global X direction (transverse to the girders) and global Z direction (vertical), allowing translation in the global Y direction (longitudinal to the girders) and rotation in all directions. Table 5 displays the support conditions of the joints at the locations of the bearing plates in local and global coordinates.

Table 5: SAP2000® Support Restraints

SAP2000® Support Restraints			
Restraint in Local Coordinates	**Restraint in Global Coordinates**	**I-End**	**J-End**
Translation 1	Translation X	Fix	Fix
Translation 2	Translation Y	Fix	Free
Translation 3	Translation Z	Fix	Fix
Rotation about 1	Rotation about X	Free	Free
Rotation about 2	Rotation about Y	Free	Free
Rotation about 3	Rotation about Z	Free	Free

3.3.5. Load Patterns

Four load patterns were defined in the SAP2000® model. The four load patterns defined were panel 1, panel 2, panel 3, and panel 4. These four load patterns represented the load applied to the girders due to the dead load of individual precast panel. These load patterns did not include the dead load of the girders. The dead load of the girders was accounted for by setting their vertical profile based on the initial survey. The initial survey represented the vertical profile of the girders already subject dead load deflections.

3.3.6. Joint Loads

A point load, in the negative Z direction, was applied to the joints at the location of each leveling screw. This point load was the average precast panel weight dispersed evenly between the eight leveling screws. The panels were loaded concentric to the min-span of the girders to produce the greatest deflections. Because of this loading pattern, and the dimensions of the four precast panels, all of the joint loads are positioned concentric to the middle of the girders and do not span the entire length of the girders.

Figure 26 displays the joint loads for the leveling screws of all precast panels condensed towards the middle of the girder span.

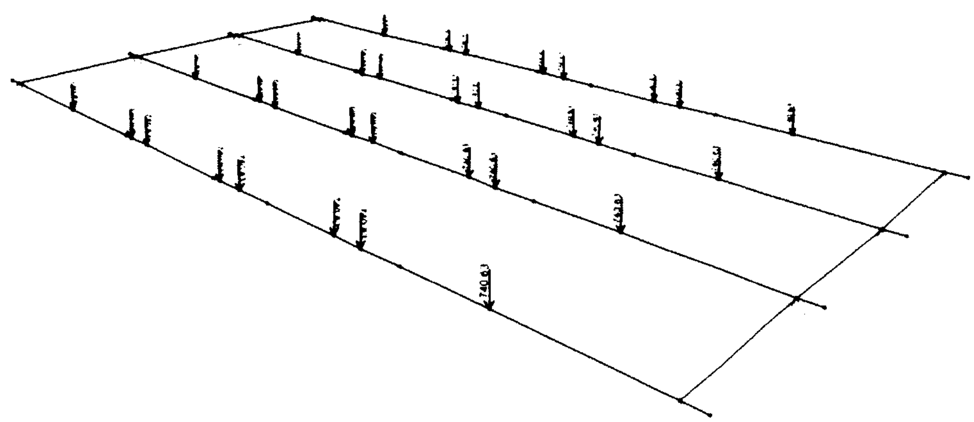

Figure 26: SAP2000 Model - All Joint Loads

These joint loads were assigned to their corresponding load pattern depending on the leveling screw load they represented. Each load pattern represented the dead load applied to the girders from a single panel and consisted of the eight leveling screw loads of that panel. The joint loads representing the axial loads of the leveling screws for panel 1 is displayed in Figure 27.

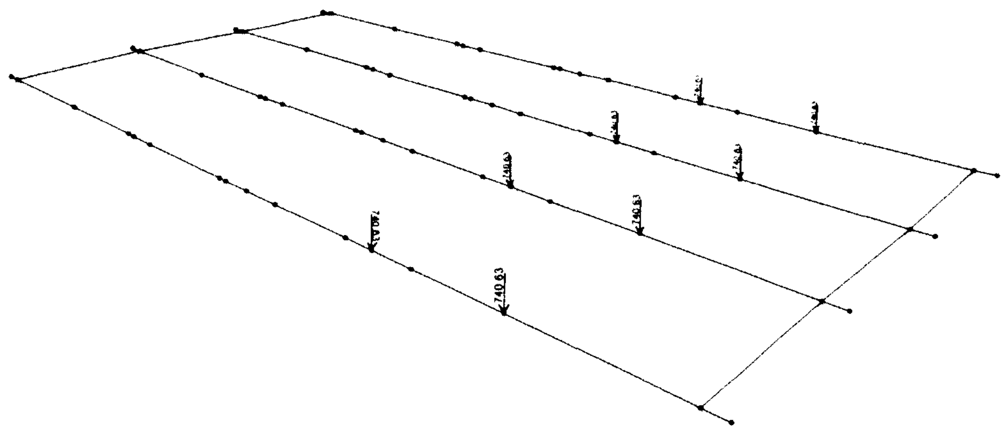

Figure 27: Panel 1 Joint Loads

The joint loads representing the axial loads of the leveling screws for panel 2 is displayed in Figure 28.

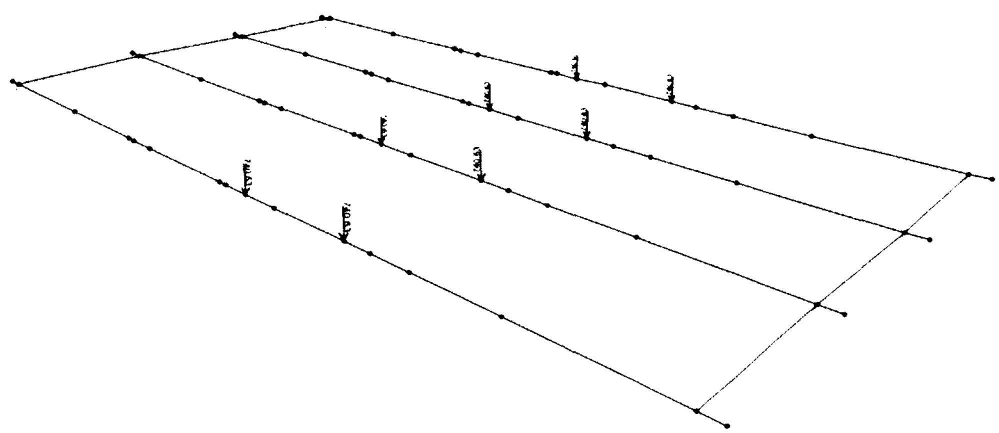

Figure 28: Panel 2 Joint Loads

The joint loads representing the axial loads of the leveling screws for panel 3 is displayed in Figure 29.

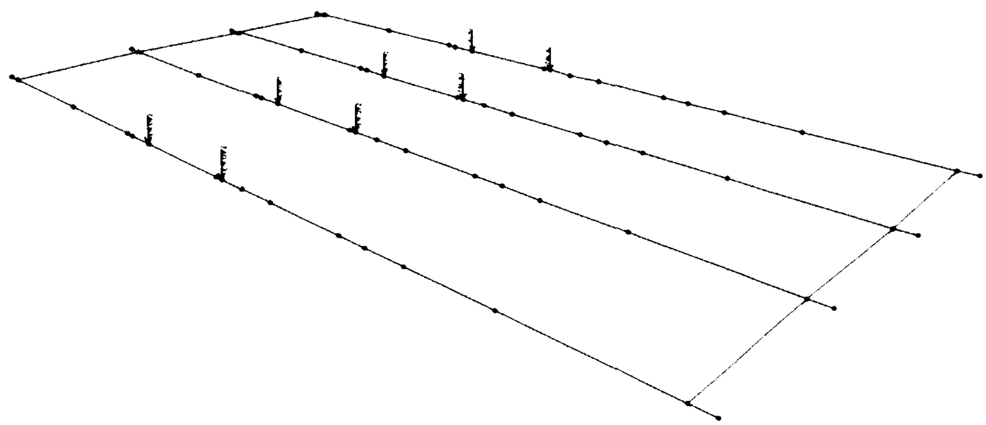

Figure 29: Panel 3 Joint Loads

The joint loads representing the axial loads of the leveling screws for panel 4 is displayed in Figure 30.

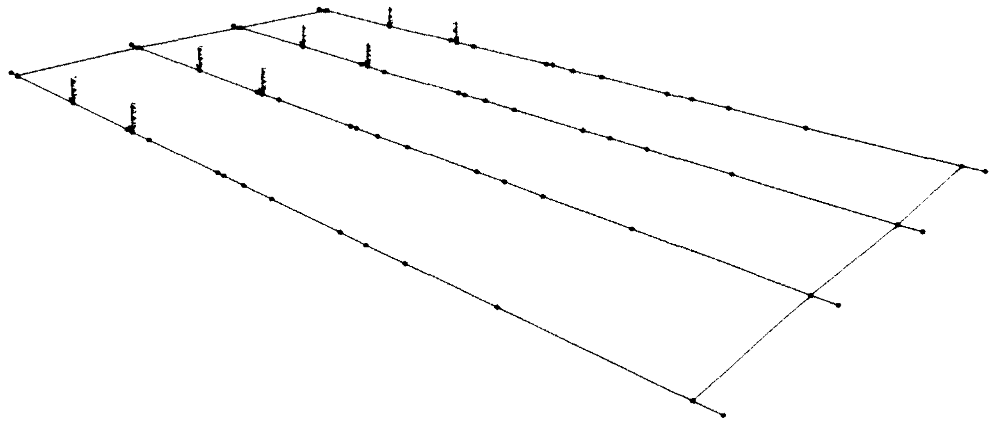

Figure 30: Panel 4 Joint Loads

3.3.7. Load Cases

Once the load patterns were defined and joint loads of each individual panel assigned to the respective load pattern, the load cases for the model were defined. The four load cases for this model were defined based on the loading configuration of the precast panel staged construction sequence. The panels were placed consecutively, one after the other, from the I-End of the girders to the J-end. In order to replicate this construction sequence, a load case was defined for each stage of loading. These load cases were defined as 1 panel, 2 panels, 3 panels, and 4 panels. These load cases represent the number of panels placed on the girders during that stage of assembly. The load cases were constructed by combining the load patterns of the respective panels and applying those loads simultaneously to the girders. The 1 panel load case contained only the panel 1 load pattern. This load case represented the dead load of just the first panel. The 2 panels load case contained both the panel 1 and panel 2 load pattern. This load case represented the dead load of the first panel as well as the addition of the dead load of the second panel. The 3 panels load case contained the panel 1, panel 2, and panel 3 load pattern. This load case represented the dead load of the first, second and third panel. The 4 panels load case contained the panel 1, panel 2, panel 3 and panel 4 load pattern. This load case represented the dead load of the first, second, third and fourth panel.

The 1 panel load case includes the joint loads for the first panel only as seen in Figure 31.

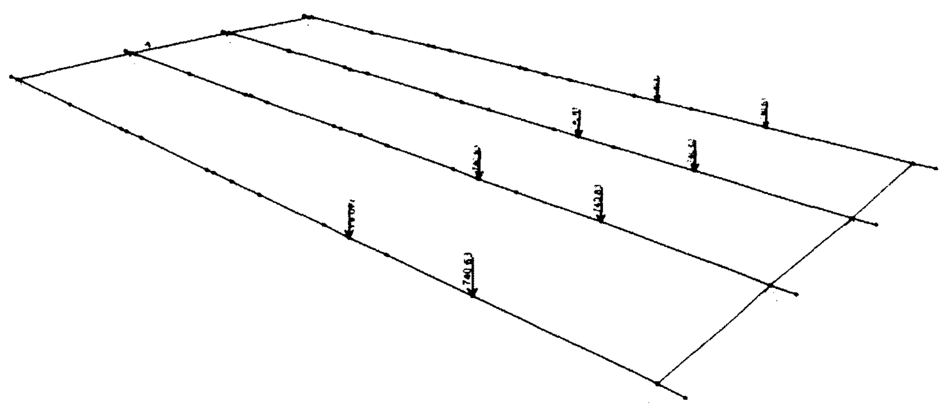

Figure 31: Load Case 1 Panel

The 2 panels load case includes the joint loads from panel 1 and panel 2 as seen in Figure 32.

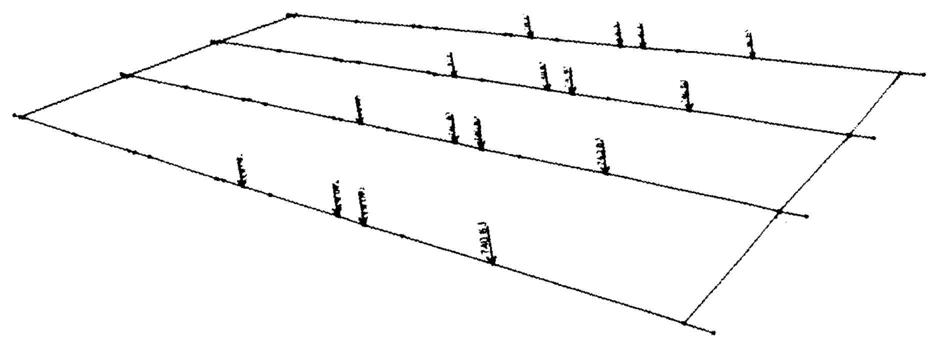

Figure 32: Load Case 2 Panels

The 3 panels load case includes the joint loads from panel 1, panel 2, and panel 3 as seen in Figure 33.

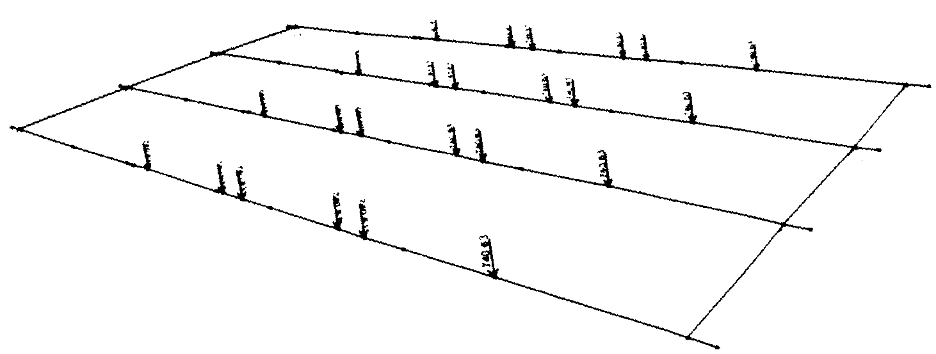

Figure 33: Load Case 3 Panels

The 4 panels load case includes the joint loads for panel 1, panel 2, panel 3, and panel 4 as seen in Figure 34.

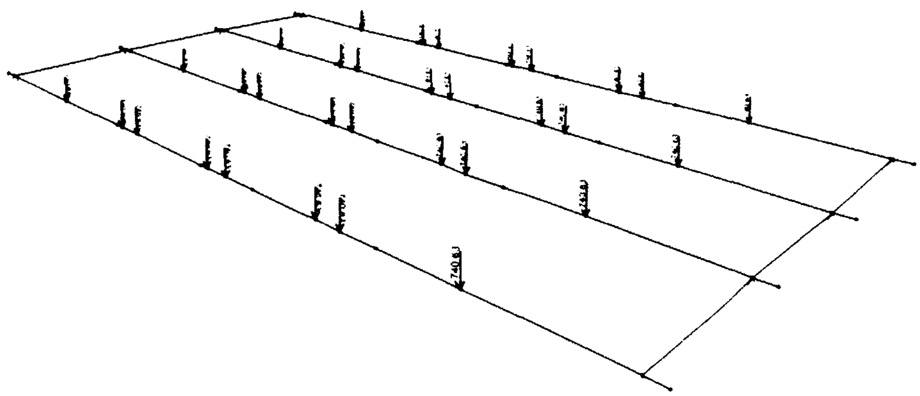

Figure 34: Load Case 4 Panels

3.3.8. Model Updating

After the girders of the four girder lab model were surveyed for existing conditions, the SAP2000® model was updated to match the elevations of the supports and the geometric shape of the girders. The SAP2000® model was updated by modifying the elevations of the joints in the model. The elevation of each joint was calculated using the equation of a trend line fit to the surveyed conditions of each girder. It was necessary to calculate the elevations of the joints of each girder to avoid surveying each location on the girder where a joint existed in the structural analysis model. If an equation of the girder geometry was not developed, each girder would require to be surveyed at 16 locations, or 64 total locations for the four girders which would increase the time necessary to complete the survey procedure.

As a part of this research a study was completed comparing the fit of polynomial trend lines in Microsoft Excel® to determine the most effective method to accurately represent the shape of the girder. Four polynomial trend lines were compared including second, third, fourth, and fifth order curves. Figure 35 displays the survey data, polynomial trend lines and their equations. These trend lines were all fit to the same set of data and their coefficients of determination (R^2) values were compared.

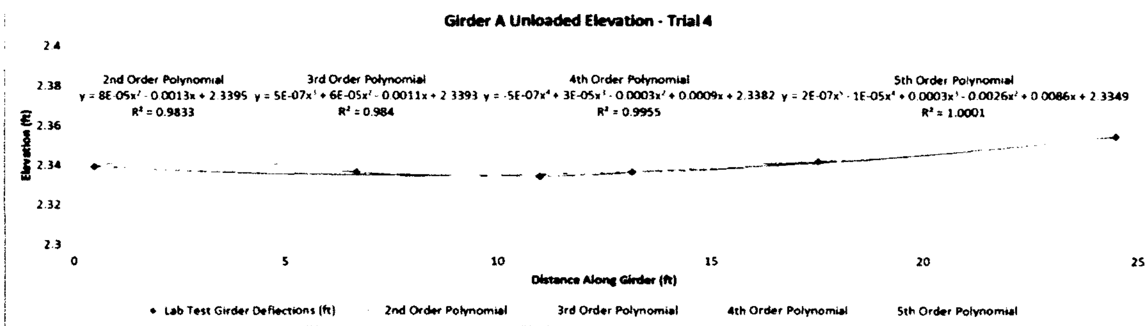

Figure 35: Polynomial Trend Line Comparison

For this survey point configuration, the fourth order polynomial curve generated the desired girder profile. The fourth order polynomial curve had the highest R^2 value without exceeding a value of 1. The fifth order curve was not used because it appeared to exaggerate the curvature of the girder more than the fourth order curve. The fourth order polynomial curve was used throughout this research as the best mathematical representation of the physical geometry of the girders.

The fourth order polynomial trend line generates a fourth order equation that can be used to calculate the elevation of the girder at any point along its length. This equation was used to generate the elevation of the girders at the location of the joints within the SAP2000® model. This was necessary to ensure the geometry of the four girder lab setup matched the SAP2000® model. Once the elevation of the girders at each joint location in SAP2000® was generated, the model was updated. This was done using the interactive database editor within SAP2000®.

The interactive database editing menu contains all of the model properties available for editing. To update the elevations of the four girder model, only the joint coordinates were edited. The frame elements were connected at the joints, so when the joint coordinates were updated, the frame elements updated to match the elevations.

3.4. Lab Trial Construction Sequence

The lab trial construction sequence consisted of the model analysis completed within SAP2000®, the calculation of the leveling screw lengths for each panel, and the sequential loading of the panels onto the girders.

3.4.1. SAP2000® Model Analysis

Each trial was treated as a staged construction process within SAP2000® with five major steps. These steps were model updating, stage 1: one panel, stage 2: two panels, stage 3: three panels and stage 4: four panels.

Model Updating Using Existing Conditions

The first step updated the girder geometry in SAP2000® to reflect the geometry of the girders in the lab using the initial condition survey data. The girder geometry from the initial conditions survey was representative of the deflection in the girders due to their own dead load. Because of this no additional dead load was applied to the girders during the analysis. Figure 36 shows the girders without any additional loads being applied to them.

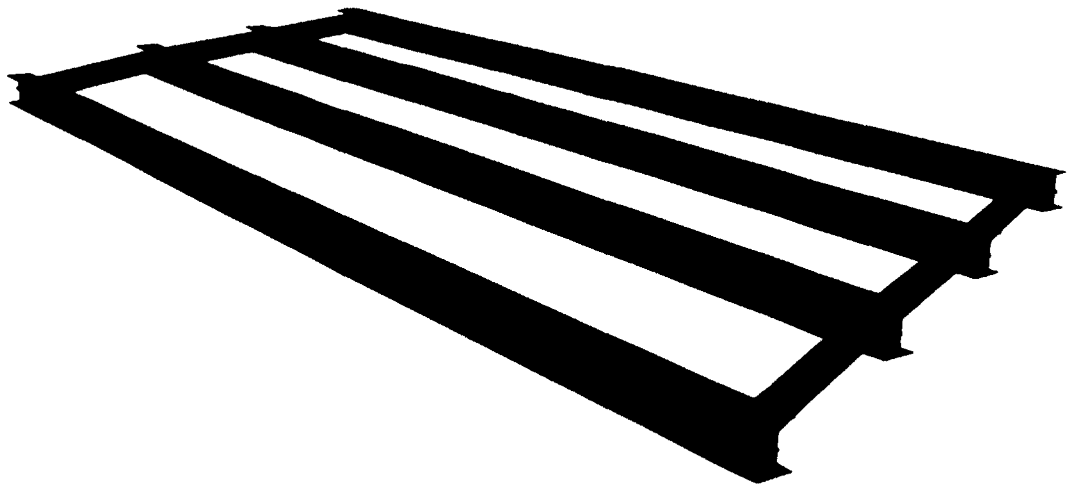

Figure 36: SAP2000® Deflected Shape No Panels

Stage 1: One Panel

The second step in the analysis was to set load case '1 panel' to run in SAP2000®. This load combination allowed for the addition of joint loads representing the axial loads of the leveling screws for panel one. Figure 37 shows the axial loads applied to the joints for the first panel.

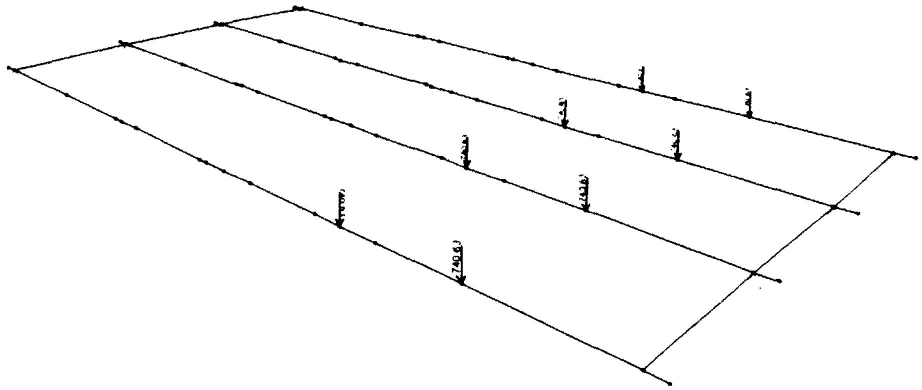

Figure 37: Set Load Case to Run 1 Panel

Load case '1 panel' was then run and the deflections of the girders were exported to Microsoft Excel® to compare to the deflections of the girders in the lab trial. Figure 38 displays the deflected shape of the girders in the '1 panel' load case.

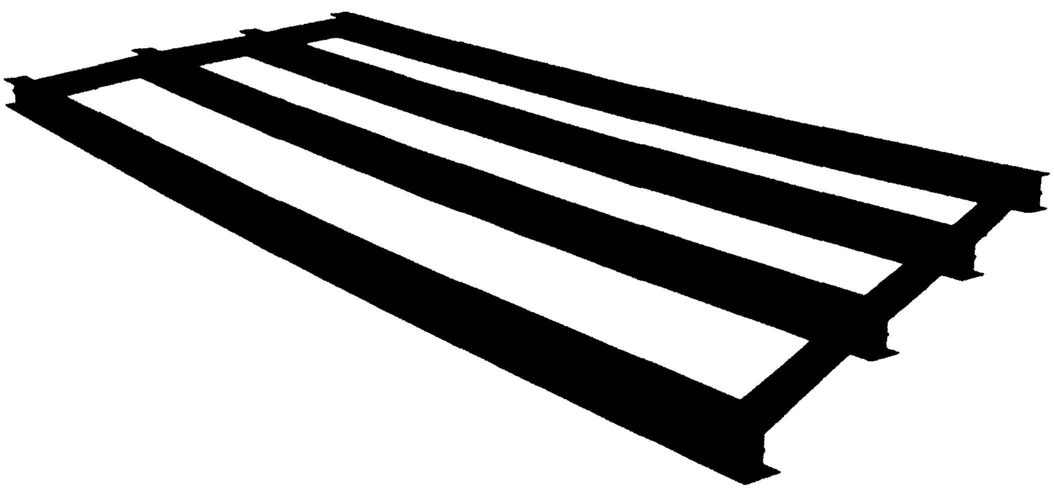

Figure 38: SAP2000® Deflected Shape 1 Panel

Stage 2: Two Panels

The third step in the analysis was to set load case '2 panels' to run in SAP2000®. This load combination allowed for the addition of joint loads representing the axial loads of the leveling screws for panel one and panel two. Figure 39 shows the axial loads applied to the joints for the first and second panel.

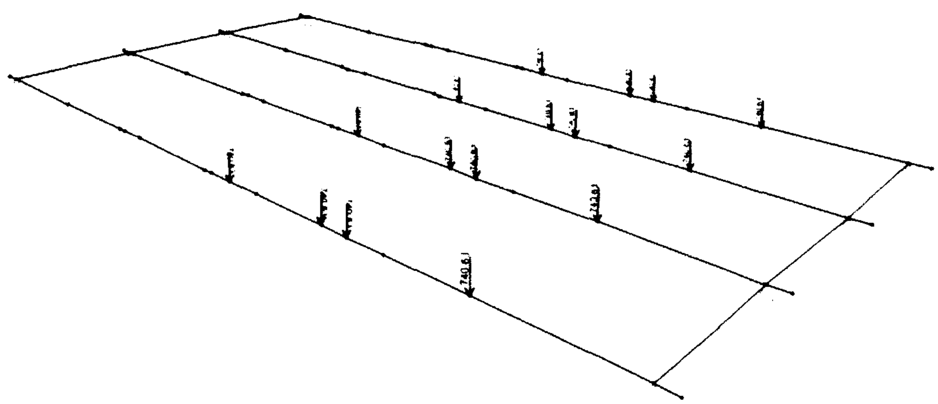

Figure 39: Set Load Case to Run 2 Panels

Load case '2 panels' was then run and the deflections of the girders were exported to Microsoft Excel® to compare to the deflections of the girders in the lab trial. Figure 40 displays the deflected shape of the girders in the '2 panels' load case.

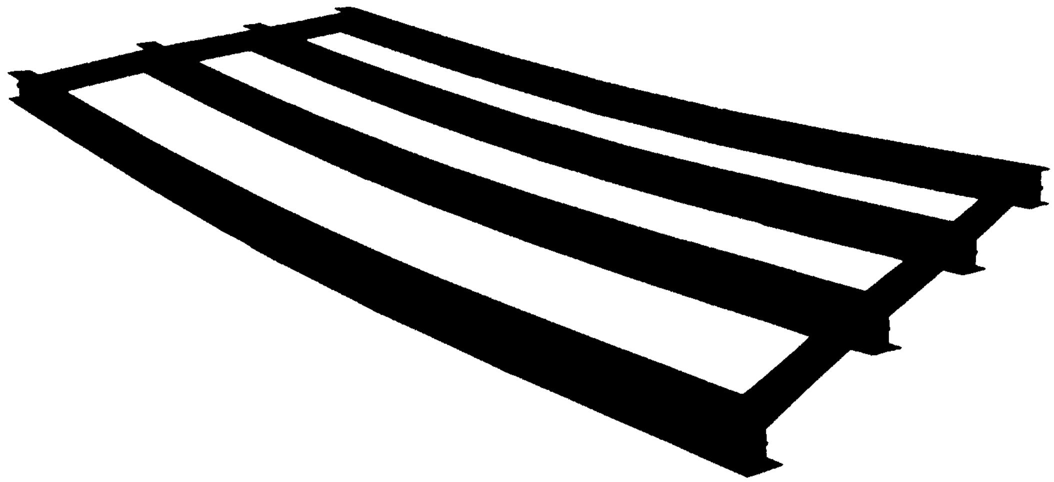

Figure 40: SAP2000® Deflected Shape 2 Panels

Stage 3: Three Panels

The fourth step in the analysis was to set load case '3 panels' to run in SAP2000®. This load combination allowed for the addition of joint loads representing the axial loads of the leveling screws for panels one, two, and three. Figure 41 shows the axial loads applied to the joints for the first, second, and third panel.

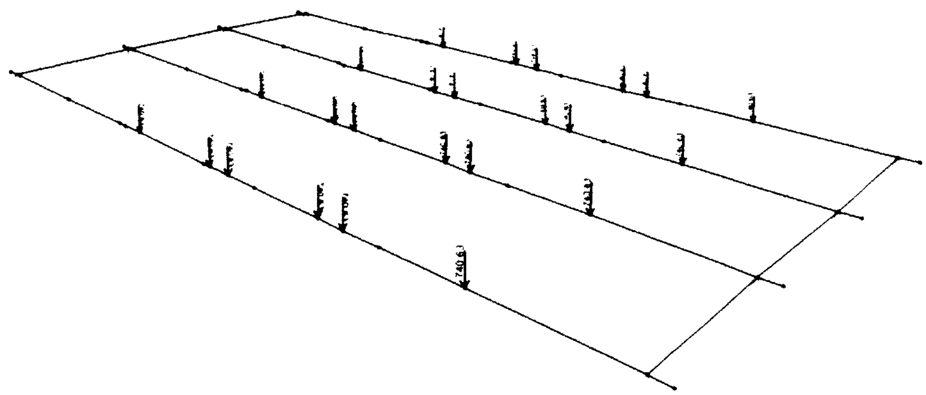

Figure 41: Set Load Case to Run 3 Panels

Load case '3 panels' was then run and the deflections of the girders were exported to Microsoft Excel® to compare to the deflections of the girders in the lab trial. Figure 42 displays the deflected shape of the girders in the '3 panels' load case.

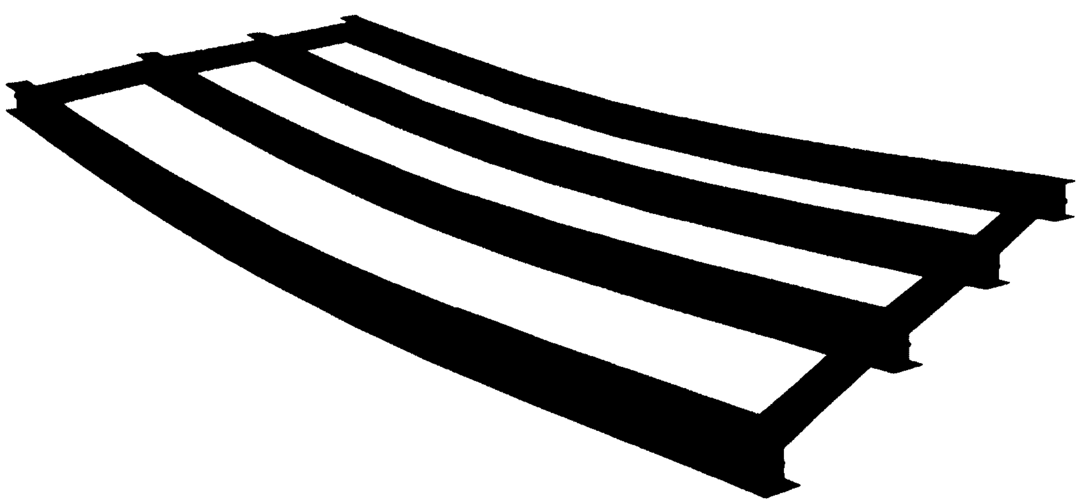

Figure 42: SAP2000® Deflected Shape 3 Panels

Stage 4: Four Panels

The fifth step in the analysis was to set load case '4 panels' to run in SAP2000®. This load combination allowed for the addition of joint loads representing the axial loads of the leveling screws for all of the panels.

Figure 43 shows the axial loads applied to the joints for all of the panels.

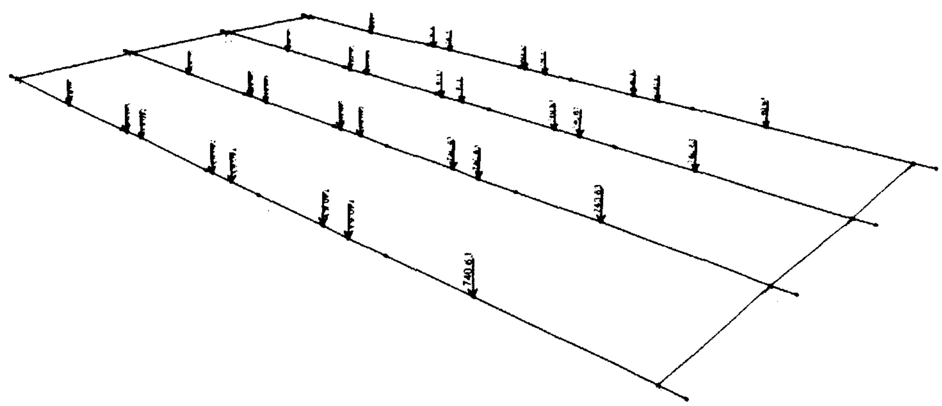

Figure 43: Set Load Case to Run 4 Panels

Load case '4 panels' was then run and the deflections of the girders were exported to Microsoft Excel® to compare to the deflections of the girders in the lab trial. Figure 44 displays the deflected shape of the girders in the '4 panels' load case.

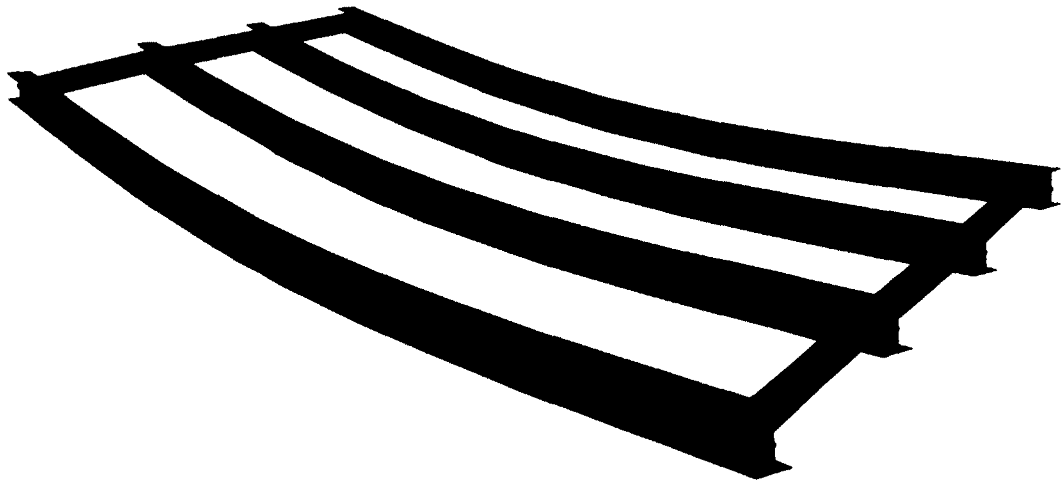

Figure 44: SAP2000® Deflected Shape 4 Panels

3.4.2. Leveling Screw Length Calculation

The length of each leveling screw is important to achieve the desired profile of the top surface of the fully installed bridge deck. The ability to determine the required length of the leveling screws prior to the installation of the precast panels eliminates the need to level the panels after they have been placed on the girders while still achieving the desired profile of the bridge deck.

The leveling screw lengths were calculated in three parts. These parts were the leveling screw length do to the thickness of the haunch, the deflection of the girders, and the profile of the bridge. Figure 45 shows the three parts of the leveling screw length calculation as well as the progression towards these values.

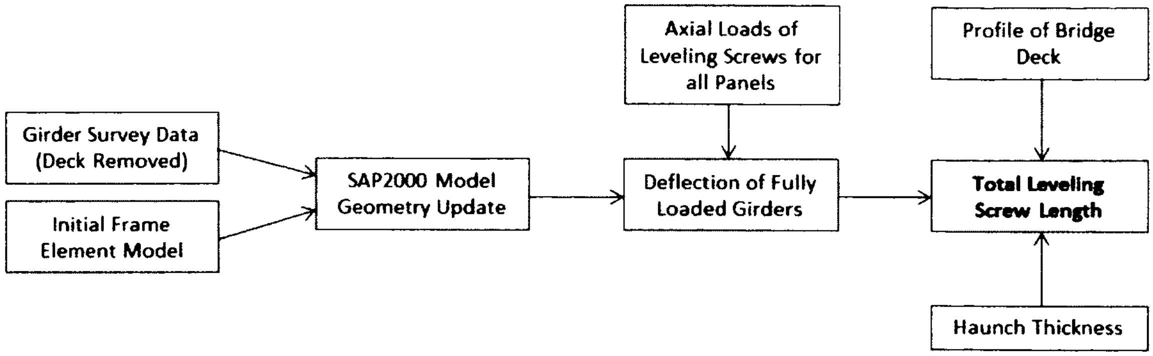

Figure 45: Leveling Screw Length Calculation

The first step in calculating the leveling screw length was to calculate the distance between the elevation of the bottom surface of the slab, at the desired elevation, and the top surface elevations of the undeflected girder. For this calculation the bottom surface of the slab was assumed to be level and the calculated girder elevations at the location of each leveling screw, from the initial conditions survey, were used as the elevations of the undeflected girders. Figure 46 shows the leveling screw length between the bottom of slab elevation and top of undeflected girder elevation.

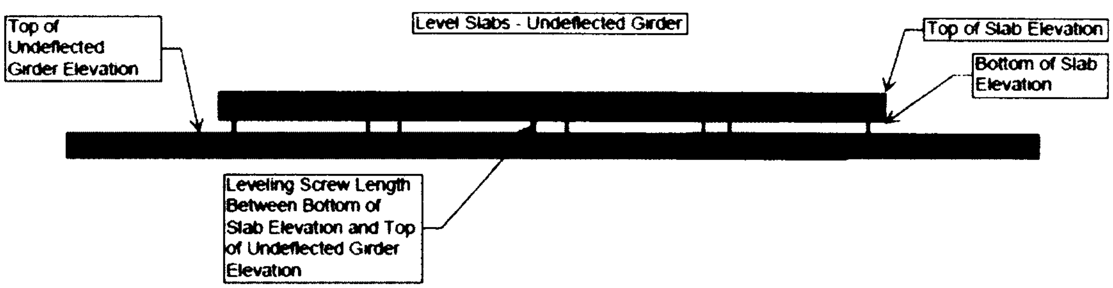

Figure 46: Level Bridge Deck Undeflected Girder Leveling Screw Length

The second step in calculating the length of the leveling screws was to calculate the distance between the top of the undeflected girder elevation and the top of the deflected girder elevation under the full dead load of all the panels. This is the distance that the girders deflected due to loading. This calculation was done by running an analysis in SAP2000®, with all four panels applied. The deflection of the girders, at the location of each leveling screw, was then exported. Figure 47 shows the leveling screw length between the top of the undeflected girder elevation and the top of the deflected girder elevation.

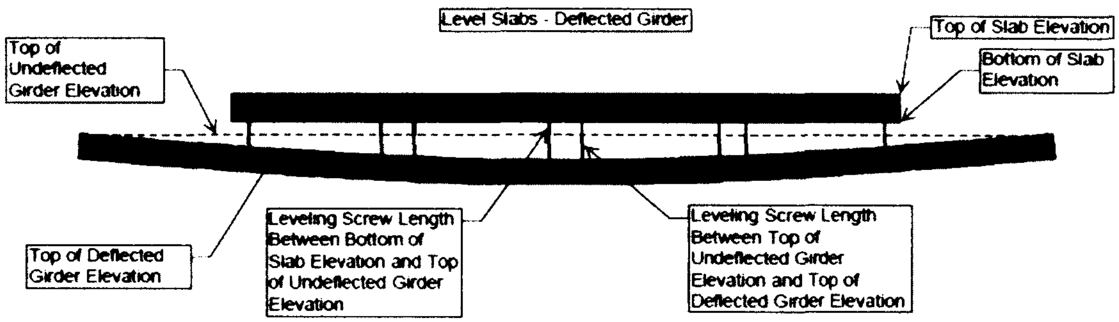

Figure 47: Level Bridge Deck Deflected Girder Leveling Screw Length

If a level bridge deck is desired, the leveling screw length between the bottom of slab elevation and the top of undeflected girder elevation would be added to the leveling screw length between the top of undeflected girder elevation and the top of deflected girder elevation to calculate the total leveling screw length.

If a sloped bridge deck is desired there is one more step in the leveling screw length calculation. The slope of the bridge deck affects the overall length of the leveling screws. This additional distance would either increase or decrease, along the longitudinal length of the bridge, with a positive or negative sloped bridge deck.

The third step of the leveling screw calculation is to account for the slope of the bridge deck. The leveling screw length due to the slope of the bridge deck is the distance between the bottom of sloped slab elevation and the bottom of level slab elevation. The elevations of the leveling screw locations can be calculated using the slope of the bridge deck and the longitudinal distance the leveling screw is from the beginning of the bridge deck. Figure 48 shows the leveling screw length between the bottom of sloped slab elevation and the bottom of level slab elevation.

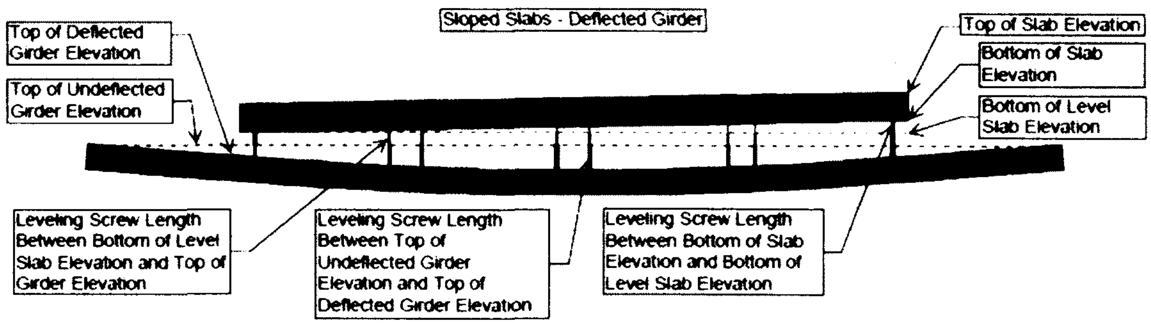

Figure 48: Sloped Bridge Deck Deflected Girder Leveling Screw Length

If a sloped bridge deck is desired, the leveling screw length between the bottom of slab elevation and the top of undeflected girder elevation would be added to the leveling screw length between the top of undeflected girder elevation and the top of deflected girder elevation. This length would then be added to the leveling screw length between the bottom of sloped slab elevation and the bottom of level slab elevation to calculate the total leveling screw length. This calculation process was used throughout this research in calculating the length of each leveling screw, from the bottom of the slab to the top of the girder, to achieve a desired final bridge deck profile.

3.4.3. Lab Trial Panel Loading Sequence

The panel loading sequence used in the lab trials differed slightly from the proposed construction sequence. The variation in sequences was due to the scope of this research. This research did not focus on the post tensioning process of the panels. The processes not included were the application of the transverse joint material, the joining of the panels so joint material squeezes out, allowing the joint material to cure, and the full post tensioning of the panels together. The panel loading sequence of this research included setting the leveling screw lengths, placing the panel on the girders, adjusting the torque of the leveling screws, and surveying the girders for deflection. This process was repeated for each panel placed on the girders.

After the initial conditions survey of the undeflected girders was completed, the SAP2000® analysis was run for all load cases and the leveling screw lengths were calculated. The leveling screw lengths were set for the first panel. The leveling screw lengths were set to the nearest $1/16^{th}$ of an inch using a standard tape measure. The leveling screw lengths were set prior to the lifting of the panel. This was because once the panel was lifted it was not possible to spin the leveling screws due to the stationary hooks being used on the strong back.

Once the leveling screw lengths were set, the first panel was lifted using a strong back and the overhead crane in the lab. The panel was then set onto the girders in a predetermined location. The location of the first panel was such that, when all four panels were loaded onto the girders, the panels would be loaded concentrically about the mid-span of the girders. This loading configuration induced the maximum deflection in the girders.

After the first panel was positioned correctly on the girders, the torque of each leveling screw was adjusted. The torque setting of each leveling screw was equal to ensure the axial load applied to each girder was equal. This was necessary because the tributary area supported by each leveling screw was equal for all of the leveling screws in each panel. If one or more of the leveling screws was not bearing on the top flange of the girder when the panel was set, these leveling screws were set to the proper torque before any others. After all of the leveling screws were bearing on the girders, the torque on each leveling screw was set in a cross pattern. The torque was then checked a second time in the same cross pattern to ensure equal torque settings on each leveling screw.

After the leveling screws were torqued equally, the girders were surveyed for deflection with an Auto Level. The surveyed deflections of the girders due to the load of one panel were then compared to the calculated deflections from SAP2000®.

After the girders were surveyed, the leveling screws of the second panel were set. The panel was lifted and set onto the girders directly abutting the first panel. There was no need to leave a space between the two panels because the application of the transverse joint material and the post tensioning of the panels was outside the scope of this research. Once the panel was set onto the girders, the leveling screws were set for torque in a cross pattern and checked a second time. The girders were then surveyed to determine their deflection due to the dead load of two panels. These deflections were then compared to the calculated deflections from SAP2000®.

This process of setting the leveling screw lengths of the panel, placing the panel onto the girders directly adjacent to the previous panel, setting the torque of the leveling screws in a cross pattern, checking the torque of the leveling crews a second time and then surveying the girders for deflection was repeated for the remaining two panels.

After the four panels were set onto the girders, and the girders had been surveyed for their fully loaded deflection, the top surface of the bridge deck was surveyed. The top surface of the bridge deck was surveyed to determine if the profile of the completed bridge deck matched the desired profile of the bridge deck in which the leveling screw lengths were calculated to match. The top surface was surveyed using an Auto Level along the two outside girders (Girder A and Girder D). The profile of the completed bridge deck, along each girder, was then compared to the desired profile used to calculate the lengths of the leveling screws.

3.5. Trial Results

Four full scale lab trials were completed in which all the panels were loaded onto the girders, in sequence, and girder deflections measured at the end of each construction stage. The process used to place the panels, as well as the SAP2000 analysis, was changed between each trial to accommodate the discovery of more accurate information and methods.

3.5.1. Trial 1

The goal of trial 1 was to predict the leveling screw length necessary to produce a level bridge deck, after all panels were placed on the girders. Another goal of this trial was to identify sources of error in the analysis as well as possible time saving procedures during assembly.

The weights of the panels were calculated using the volume of each panel and the weight per unit volume of concrete (See APPENDIX B). This was done because this trial took place before the precast panels were weighed and the actual weights determined. The calculated panel weight used for this trial was 6720 lb. and an axial load of 840 lb. was used to represent each leveling screw load. The torque applied to each of the leveling screws was determined using an empirical equation which took into account the axial load applied, diameter of the leveling screw, the thread depth and pattern, and the friction of the steel on concrete. The value obtained was roughly 140 in.-lbs. This value of torque was used for this trial because the leveling screw torque study had not yet been completed. The elevations of the girders were surveyed to the nearest sixteenth of an inch with a transit during this trial.

Model Updating Using Existing Conditions

The girders were surveyed for their elevation using a transit. The floor of the Structures Laboratory was treated as zero elevation. In this trial only the supports were surveyed and the girders were assumed to be perfectly straight. The unloaded girder elevations were calculated using a linear equation and the elevations of the supports. Table 6 shows the support elevations and the calculated elevations of the unloaded girders. After this trial it was determined that the girders were not linear as initially assumed.

Table 6: Trial 1 Unloaded Girder Elevations

Unloaded Girder Elevation (ft.)				
Distance (ft.)	Girder A	Girder B	Girder C	Girder D
0.50	2.3281	2.3333	2.3073	2.3281
6.67	2.3254	2.3320	2.3167	2.3308
11.00	2.3236	2.3311	2.3232	2.3327
13.17	2.3226	2.3306	2.3265	2.3336
17.50	2.3207	2.3296	2.3331	2.3355
24.50	2.3177	2.3281	2.3438	2.3385

The elevations of the supports were used in a linear equation to find the elevations of the top flange of the girders at the location of each leveling screw. These elevations were used to update the geometry of the girders in the SAP2000® model. The predicted deflections of the girders, under full dead load, were obtained from SAP2000® and used to calculate the length of the leveling screws. The profile of the bridge deck was set to a slope of 0.0% for this trial.

Stage 1: One Panel

The leveling screws for the first panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted to 140 in.-lbs. such that an equal torque was applied to each screw. The girders were then surveyed for their elevation and the deflections were calculated. An analysis was run in SAP2000® using the '1 panel' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 7: Trial 1 Girder Deflections - One Panel

Girder Deflections - One Panel								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0140	-0.0140	-0.0140	-0.0140	-0.0104	-0.0104	-0.0104	-0.0104
11.00	-0.0171	-0.0171	-0.0171	-0.0171	-0.0052	-0.0260	-0.0156	-0.0156
13.17	-0.0165	-0.0165	-0.0165	-0.0165	-0.0104	-0.0260	-0.0156	-0.0156
17.50	-0.0122	-0.0122	-0.0122	-0.0122	-0.0052	-0.0156	-0.0156	-0.0104

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 8: Trial 1 Girder Deflection Differences - One Panel

Differences between SAP2000® and Lab Test Girder Deflection - One Panel								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0036	0.0036	0.0036	0.0036	-35%	-35%	-35%	-35%
11.00	0.0119	-0.0089	0.0015	0.0015	-229%	34%	-10%	-10%
13.17	0.0061	-0.0095	0.0009	0.0009	-59%	37%	-6%	-6%
17.50	0.0070	-0.0035	-0.0035	0.0017	-134%	22%	22%	-17%

Stage 2: Two Panels

The leveling screws for the second panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted to 140 in.-lbs. such that an equal torque was applied to each screw. The girders were then surveyed for their elevation and the deflections were calculated. An analysis was run in SAP2000® using the '2 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 9: Trial 1 Girder Deflections - Two Panels

Girder Deflections - Two Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0316	-0.0316	-0.0316	-0.0316	-0.0208	-0.0313	-0.0365	-0.0313
11.00	-0.0406	-0.0406	-0.0406	-0.0406	-0.0313	-0.0417	-0.0365	-0.0365
13.17	-0.0399	-0.0399	-0.0399	-0.0399	-0.0313	-0.0469	-0.0417	-0.0417
17.50	-0.0299	-0.0299	-0.0299	-0.0299	-0.0260	-0.0365	-0.0313	-0.0313

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 10: Trial 1 Girder Deflection Differences - Two Panels

Differences between SAP2000® and Lab Test Girder Deflection - Two Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0108	0.0004	-0.0048	0.0004	-52%	-1%	13%	-1%
11.00	0.0093	-0.0011	0.0041	0.0041	-30%	3%	-11%	-11%
13.17	0.0086	-0.0070	-0.0018	-0.0018	-28%	15%	4%	4%
17.50	0.0039	-0.0065	-0.0013	-0.0013	-15%	18%	4%	4%

Stage 3: Three Panels

The leveling screws for the third panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted to 140 in.-lbs. such that an equal torque was applied to each screw. The girders were then surveyed for their elevation and the deflections were calculated. An analysis was run in SAP2000® using the '3 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 11: Trial 1 Girder Deflections - Three Panels

Girder Deflections - Three Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0475	-0.0475	-0.0475	-0.0475	-0.0417	-0.0469	-0.0521	-0.0469
11.00	-0.0632	-0.0632	-0.0632	-0.0632	-0.0521	-0.0729	-0.0677	-0.0625
13.17	-0.0634	-0.0634	-0.0634	-0.0634	-0.0573	-0.0677	-0.0677	-0.0677
17.50	-0.0491	-0.0490	-0.0490	-0.0491	-0.0417	-0.0521	-0.0469	-0.0469

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 12: Trial 1 Girder Deflection Differences - Three Panels

Differences between SAP2000® and Lab Test Girder Deflection - Three Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0058	0.0006	-0.0046	0.0006	-14%	-1%	9%	-1%
11.00	0.0111	-0.0097	-0.0045	0.0007	-21%	13%	7%	-1%
13.17	0.0061	-0.0043	-0.0043	-0.0043	-11%	6%	6%	6%
17.50	0.0074	-0.0030	0.0022	0.0022	-18%	6%	-5%	-5%

Stage 4: Four Panels

The leveling screws for the fourth panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted to 140 in-lbs. such that an equal torque was applied to each screw. The girders were then surveyed for their elevation and the deflections were calculated. An analysis was run in SAP2000® using the '4 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 13: Trial 1 Girder Deflections - Four Panels

Girder Deflections - Four Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0584	-0.0584	-0.0584	-0.0584	-0.0625	-0.0677	-0.0677	-0.0625
11.00	-0.0790	-0.0790	-0.0790	-0.0790	-0.0771	-0.0885	-0.0833	-0.0781
13.17	-0.0803	-0.0803	-0.0803	-0.0803	-0.0771	-0.0885	-0.0781	-0.0833
17.50	-0.0640	-0.0640	-0.0640	-0.0641	-0.0677	-0.0625	-0.0573	-0.0573

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 14: Trial 1 Girder Deflection Differences - Four Panels

Differences between SAP2000® and Lab Test Girder Deflection - Four Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0041	-0.0093	-0.0093	-0.0041	7%	14%	14%	7%
11.00	0.0019	-0.0095	-0.0043	0.0009	-3%	11%	5%	-1%
13.17	0.0032	-0.0083	0.0021	-0.0031	-4%	9%	-3%	4%
17.50	-0.0037	0.0015	0.0067	0.0067	5%	-2%	-12%	-12%

Trial 1 identified many areas in which changes could be made to the process to improve accuracy and ease of construction. The main areas of error were identified as the survey procedure and the girder geometry assumptions.

Surveying the girders with a transit was time consuming. The transit was constantly in need of adjustment and leveling. It was decided after this trial that the survey should be completed with a device that provides higher accuracy and ease of use, such as an Auto Level.

Another discovery was that the girders could not be assumed to be perfectly straight. It was obvious, with the placement of each panel, that the girders had a small degree of curvature. This curvature caused some of the leveling screws to not bear on the girders directly after panel placement. The curvature of the girders should be accounted for in future trials.

3.5.2. Trial 2

Trial 2 was completed in order to address some of the sources of error identified in trial 1. Instead of assuming the girders were linear, each girder was surveyed for its elevation in the six locations discussed earlier in this report. (Section 3.2.2) The survey was also done using an Auto Level instead of a transit to increase the accuracy of the survey data. These elevations were used to determine the vertical profile of the girder to be used in the SAP2000® analysis. The torqueing pattern of the leveling screws was changed to a cross pattern instead of a random pattern. The torque of the leveling screws was also adjusted twice during this trial instead of once. Another goal of trial 2 was to introduce a sloped bridge deck into the leveling screw length calculation.

The weight of the panels was calculated using the volume of each panel and the weight per unit volume of concrete. This trail, along with trial 1, also occurred prior to the weighing of each panel. An axial load of 840 lb. was used to represent the leveling screw load. After each panel was placed the leveling screws were torqued to 140 in.-lbs. in a random cross pattern, and then checked a second time, to evenly distribute the dead load to the girders. This value of torque was used for this trial because the leveling screw torque study had not yet been completed. The elevations of the girders were surveyed to the nearest hundredth of a foot during this trial.

Model Updating Using Existing Conditions

The girders were surveyed for their elevation with an Auto Level. Before the survey, the elevations of the supports were altered using shims to create different girder profiles from that of trial 1. The floor of the Structures Laboratory was treated as zero elevation. In this trial, the girders were surveyed in six locations including each support. The survey was used to calculate the elevation of each girder at these six locations.

Table 15: Trial 2 Unloaded Girder Elevations

Unloaded Girder Elevation (ft.)				
Distance (ft.)	Girder A	Girder B	Girder C	Girder D
0.50	2.4650	2.4400	2.4700	2.4700
6.67	2.3200	2.3700	2.3500	2.3400
11.00	2.3150	2.3700	2.3650	2.3400
13.17	2.3100	2.3720	2.3700	2.3400
17.50	2.3100	2.3800	2.3900	2.3500
24.50	2.4800	2.4100	2.3800	2.4400

These elevations were fit to a fourth order polynomial trend line and its equation was used to update the geometry of the girders in the SAP2000® model. The calculated deflections of the girders, under full dead load, were obtained from SAP2000® and used to calculate the length of the leveling screws. The profile of the bridge deck was set to a slope of -1.0% for this trial.

Stage 1: One Panel

The leveling screws for the first panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '1 panel' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 16: Trial 2 Girder Deflections - One Panel

Girder Deflections - One Panel								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0154	-0.0154	-0.0154	-0.0154	-0.0100	-0.0250	-0.0150	-0.0100
11.00	-0.0189	-0.0189	-0.0188	-0.0189	-0.0150	-0.0200	-0.0150	-0.0100
13.17	-0.0182	-0.0182	-0.0182	-0.0182	-0.0100	-0.0220	-0.0150	-0.0100
17.50	-0.0134	-0.0134	-0.0134	-0.0134	-0.0050	-0.0150	-0.0150	-0.0100

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 17: Trial 2 Girder Deflection Differences - One Panel

Differences between SAP2000® and Lab Test Girder Deflection - One Panel								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0054	-0.0096	0.0004	0.0054	-54%	38%	-3%	-54%
11.00	0.0039	-0.0011	0.0038	0.0089	-26%	6%	-26%	-89%
13.17	0.0082	-0.0038	0.0032	0.0082	-82%	17%	-21%	-82%
17.50	0.0084	-0.0016	-0.0016	0.0034	-168%	11%	11%	-34%

Stage 2: Two Panels

The leveling screws for the second panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '2 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 18: Trial 2 Girder Deflections - Two Panels

Girder Deflections - Two Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0358	-0.0358	-0.0358	-0.0358	-0.0250	-0.0450	-0.0350	-0.0300
11.00	-0.0461	-0.0461	-0.0460	-0.0461	-0.0350	-0.0500	-0.0450	-0.0350
13.17	-0.0453	-0.0453	-0.0452	-0.0453	-0.0300	-0.0470	-0.0400	-0.0320
17.50	-0.0340	-0.0340	-0.0339	-0.0340	-0.0200	-0.0350	-0.0300	-0.0300

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 19: Trial 2 Girder Deflection Differences - Two Panels

Differences between SAP2000® and Lab Test Girder Deflection - Two Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0108	-0.0092	0.0008	0.0058	-43%	20%	-2%	-19%
11.00	0.0111	-0.0039	0.0010	0.0111	-32%	8%	-2%	-32%
13.17	0.0153	-0.0017	0.0052	0.0133	-51%	4%	-13%	-41%
17.50	0.0140	-0.0010	0.0039	0.0040	-70%	3%	-13%	-13%

Stage 3: Three Panels

The leveling screws for the third panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '3 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 20: Trial 2 Girder Deflections - Three Panels

Girder Deflections - Three Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0544	-0.0544	-0.0544	-0.0544	-0.0500	-0.0600	-0.0500	-0.0500
11.00	-0.0724	-0.0724	-0.0724	-0.0725	-0.0650	-0.0750	-0.0650	-0.0600
13.17	-0.0727	-0.0727	-0.0726	-0.0727	-0.0550	-0.0720	-0.0600	-0.0600
17.50	-0.0563	-0.0563	-0.0562	-0.0563	-0.0400	-0.0550	-0.0500	-0.0500

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 21: Trial 2 Girder Deflection Differences - Three Panels

Differences between SAP2000® and Lab Test Girder Deflection - Three Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0044	-0.0056	0.0044	0.0044	-9%	9%	-9%	-9%
11.00	0.0074	-0.0026	0.0074	0.0125	-11%	3%	-11%	-21%
13.17	0.0177	0.0007	0.0126	0.0127	-32%	-1%	-21%	-21%
17.50	0.0163	0.0013	0.0062	0.0063	-41%	-2%	-12%	-13%

Stage 4: Four Panels

The leveling screws for the fourth panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '4 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 22: Trial 2 Girder Deflections - Four Panels

Girder Deflections - Four Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0665	-0.0665	-0.0665	-0.0665	-0.0550	-0.0750	-0.0600	-0.0650
11.00	-0.0901	-0.0901	-0.0900	-0.0901	-0.0730	-0.0900	-0.0800	-0.0750
13.17	-0.0915	-0.0915	-0.0914	-0.0915	-0.0700	-0.0920	-0.0800	-0.0750
17.50	-0.0730	-0.0730	-0.0729	-0.0730	-0.0520	-0.0700	-0.0620	-0.0640

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 23: Trial 2 Girder Deflection Differences - Four Panels

Differences between SAP2000® and Lab Test Girder Deflection - Four Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0115	-0.0085	0.0065	0.0015	-21%	11%	-11%	-2%
11.00	0.0171	0.0001	0.0100	0.0151	-23%	0%	-12%	-20%
13.17	0.0215	-0.0005	0.0114	0.0165	-31%	1%	-14%	-22%
17.50	0.0210	0.0030	0.0109	0.0090	-40%	-4%	-18%	-14%

Final Profile of Bridge Deck

After all four of the panels were set onto the girders, the top surface of the bridge deck was surveyed. This survey was completed to determine if the constructed profile of the bridge deck matched the desired profile. Figure 49 shows the profile of the bridge deck along girders A and D and the desired profile of -1%.

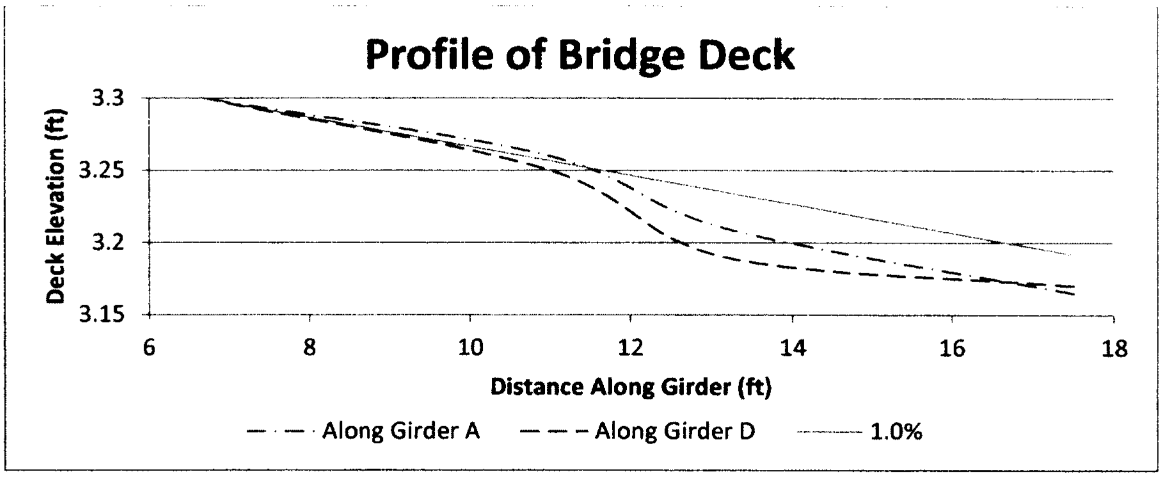

Figure 49: Trial 2 Bridge Deck Profile

The maximum difference between the actual profile and the desired profile of the bridge deck for trial 2 was 0.0139' or about 5/32nd of an inch.

The results from trial 2 showed that the girder deflections from SAP2000® were higher than the actual deflections of the girders with the exception of girder B. This implied that the loads being applied to the structural analysis model could be larger than the actual loads being applied to the girders. It was decided, based on the results of this trial, that a more accurate value for the weight of each panel was necessary.

3.5.3. Trial 3

Trial 3 was an analysis that was completed after the actual panel weights were calculated. This analysis used the lab test initial conditions and girder deflections from trial 2 and an updated SAP2000® model with adjusted leveling screw loads. The purpose of this trial was to discover the affect that the calculated weight of the panels would have on the accuracy of the girder deflection results from trial 2. The magnitude of the joint loads, representing the axial loads of the leveling screws, in the SAP2000® analysis was the only difference between trial 2 and trial 3.

Model Updating Using Existing Conditions

The girder elevations from trial 2 were used as the unloaded girder elevations. In this trial, the measured average weight of the panels, 5925 lb., was used when calculating the axial loads of the leveling screws.

Table 24: Trial 3 Unloaded Girder Elevations

Unloaded Girder Elevation (ft.)				
Distance (ft.)	Girder A	Girder B	Girder C	Girder D
0.50	2.4650	2.4400	2.4700	2.4700
6.67	2.3200	2.3700	2.3500	2.3400
11.00	2.3150	2.3700	2.3650	2.3400
13.17	2.3100	2.3720	2.3700	2.3400
17.50	2.3100	2.3800	2.3900	2.3500
24.50	2.4800	2.4100	2.3800	2.4400

These elevations were fit to a fourth order polynomial equation and used to update the geometry of the girders in the SAP2000® model. The calculated deflections of the girders, under full dead load, were obtained from SAP2000® and used to calculate the length of the leveling screws.

Stage 1: One Panel

The girder deflections from trial 2, for one panel being placed, were used. An analysis was run in SAP2000® using the '1 panel' load case which included the updated leveling screw axial loads. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 25: Trial 3 Girder Deflections - One Panel

Girder Deflections - One Panel								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0140	-0.0140	-0.0140	-0.0140	-0.0100	-0.0250	-0.0150	-0.0100
11.00	-0.0171	-0.0171	-0.0171	-0.0171	-0.0150	-0.0200	-0.0150	-0.0100
13.17	-0.0165	-0.0165	-0.0165	-0.0165	-0.0100	-0.0220	-0.0150	-0.0100
17.50	-0.0122	-0.0122	-0.0122	-0.0122	-0.0050	-0.0150	-0.0150	-0.0100

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 26: Trial 3 Girder Deflection Differences - One Panel

Differences between SAP2000® and Lab Test Girder Deflection - One Panel								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0040	-0.0110	-0.0010	0.0040	-40%	44%	7%	-40%
11.00	0.0021	-0.0029	0.0021	0.0071	-14%	14%	-14%	-71%
13.17	0.0065	-0.0055	0.0015	0.0065	-65%	25%	-10%	-65%
17.50	0.0072	-0.0028	-0.0028	0.0022	-143%	19%	19%	-22%

Stage 2: Two Panels

The girder deflections from trial 2, for two panels being placed, were used. An analysis was run in SAP2000® using the '2 panel' load case which included the updated leveling screw axial loads. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 27: Trial 3 Girder Deflections - Two Panels

Girder Deflections - Two Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0316	-0.0316	-0.0316	-0.0316	-0.0250	-0.0450	-0.0350	-0.0300
11.00	-0.0406	-0.0406	-0.0406	-0.0406	-0.0350	-0.0500	-0.0450	-0.0350
13.17	-0.0399	-0.0399	-0.0399	-0.0399	-0.0300	-0.0470	-0.0400	-0.0320
17.50	-0.0299	-0.0299	-0.0299	-0.0299	-0.0200	-0.0350	-0.0300	-0.0300

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 28: Trial 3 Girder Deflection Differences - Two Panels

Differences between SAP2000® and Lab Test Girder Deflection - Two Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0066	-0.0134	-0.0034	0.0016	-26%	30%	10%	-5%
11.00	0.0056	-0.0094	-0.0044	0.0056	-16%	19%	10%	-16%
13.17	0.0099	-0.0071	-0.0001	0.0079	-33%	15%	0%	-25%
17.50	0.0099	-0.0051	-0.0001	-0.0001	-50%	15%	0%	0%

Stage 3: Three Panels

The girder deflections from trial 2, for three panels being placed, were used. An analysis was run in SAP2000® using the '3 panel' load case which included the updated leveling screw axial loads. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 29: Trial 3 Girder Deflections - Three Panels

Girder Deflections - Three Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0475	-0.0475	-0.0475	-0.0475	-0.0500	-0.0600	-0.0500	-0.0500
11.00	-0.0632	-0.0632	-0.0632	-0.0632	-0.0650	-0.0750	-0.0650	-0.0600
13.17	-0.0634	-0.0634	-0.0634	-0.0634	-0.0550	-0.0720	-0.0600	-0.0600
17.50	-0.0491	-0.0490	-0.0490	-0.0491	-0.0400	-0.0550	-0.0500	-0.0500

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 30: Trial 3 Girder Deflection Differences - Three Panels

Differences between SAP2000® and Lab Test Girder Deflection - Three Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0025	-0.0125	-0.0025	-0.0025	5%	21%	5%	5%
11.00	-0.0018	-0.0118	-0.0018	0.0032	3%	16%	3%	-5%
13.17	0.0084	-0.0086	0.0034	0.0034	-15%	12%	-6%	-6%
17.50	0.0091	-0.0060	-0.0010	-0.0009	-23%	11%	2%	2%

Stage 4: Four Panels

The girder deflections from trial 2, for four panels being placed, were used. An analysis was run in SAP2000® using the '4 panel' load case which included the updated leveling screw axial loads. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 31: Trial 3 Girder Deflections - Four Panels

Girder Deflections - Four Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0584	-0.0584	-0.0584	-0.0584	-0.0550	-0.0750	-0.0600	-0.0650
11.00	-0.0790	-0.0790	-0.0790	-0.0790	-0.0730	-0.0900	-0.0800	-0.0750
13.17	-0.0803	-0.0803	-0.0803	-0.0803	-0.0700	-0.0920	-0.0800	-0.0750
17.50	-0.0640	-0.0640	-0.0640	-0.0641	-0.0520	-0.0700	-0.0620	-0.0640

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 32: Trial 3 Girder Deflection Differences - Four Panels

Differences between SAP2000® and Lab Test Girder Deflection - Four Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	0.0034	-0.0166	-0.0016	-0.0066	-6%	22%	3%	10%
11.00	0.0060	-0.0110	-0.0010	0.0040	-8%	12%	1%	-5%
13.17	0.0103	-0.0117	0.0003	0.0053	-15%	13%	0%	-7%
17.50	0.0120	-0.0060	0.0020	0.0001	-23%	9%	-3%	0%

This analysis proved that the actual weight of the precast panels, when used to determine the axial loads of the leveling screws, did have an impact on the difference in deflection between the SAP2000® model and the laboratory model when fully loaded. The difference in deflection between the two models was lower in trial 3 than it was in trial 2.

3.5.4. Trial 4

Trial 4 combined all of the changes made throughout the previous trials. These changes included using an Auto Level for surveying, surveying the girders in six locations to determine their vertical profile, using the actual weight of the precast panels for the SAP2000® analysis, implementing a cross pattern for torqueing the leveling screws, and adjusting the torque of the leveling screws twice. Trial 4 also included the updated leveling screw torque value determined from the leveling screw torque study. The accuracy of the survey was also increased for trial 4. The elevations of the girders were measured to the nearest thirty-second on an inch during this trial.

The axial load applied to the girders by each leveling screw was calculated using the average measured weights of the panels. The axial load applied to the joints was 740.63 lb. in the negative Z direction. The torque applied to each leveling screw was calculated using the results from the leveling screw torque study mentioned earlier. The required torque of each leveling screw was calculated to be 103.46 in.-lbs. The actual torque applied to each leveling screw in this trial was 120 in-lbs. This was the result of incorrectly setting the torque wrench during the assembly process. It was not anticipated that this discrepancy in leveling screw torque resulted in a significant error within this lab trial.

Model Updating Using Existing Conditions

The girders were surveyed for their elevation using an Auto Level. The floor of the Structures Laboratory was treated as zero elevation. In this trial, the support elevations were altered using steel shims to create more variability between trials.

Table 33: Trial 4 Unloaded Girder Elevations

Unloaded Girder Elevation (ft.)				
Distance (ft.)	Girder A	Girder B	Girder C	Girder D
0.50	2.3385	2.3646	2.3125	2.3802
6.67	2.3359	2.3750	2.3385	2.3776
11.00	2.3333	2.3802	2.3516	2.3724
13.17	2.3359	2.3802	2.3568	2.3698
17.50	2.3411	2.3880	2.3724	2.3646
24.50	2.3542	2.3984	2.4010	2.3594

These elevations were fit to a fourth order polynomial trend line and its equation was used to update the geometry of the girders in the SAP2000® model. The calculated deflections of the girders, under full dead load, were obtained from SAP2000® and used to calculate the length of the leveling screws. The profile of the bridge deck was set to a slope of -1.4% for this trial.

Stage 1: One Panel

The leveling screws for the first panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '1 panel' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 34: Trial 4 Girder Deflections – One Panel

Girder Deflections - One Panel								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0136	-0.0136	-0.0136	-0.0136	-0.0182	-0.0286	-0.0208	-0.0234
11.00	-0.0166	-0.0166	-0.0166	-0.0166	-0.0208	-0.0286	-0.0208	-0.0234
13.17	-0.0161	-0.0161	-0.0161	-0.0161	-0.0208	-0.0260	-0.0182	-0.0208
17.50	-0.0118	-0.0118	-0.0118	-0.0118	-0.0182	-0.0208	-0.0130	-0.0182

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 35: Trial 4 Girder Deflection Differences - One Panel

Differences between SAP2000® and Lab Test Girder Deflection - One Panel								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0046	-0.0150	-0.0072	-0.0098	25%	52%	35%	42%
11.00	-0.0042	-0.0120	-0.0042	-0.0068	20%	42%	20%	29%
13.17	-0.0048	-0.0100	-0.0022	-0.0048	23%	38%	12%	23%
17.50	-0.0064	-0.0090	-0.0012	-0.0064	35%	43%	9%	35%

Stage 2: Two Panels

The leveling screws for the second panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '2 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 36: Trial 4 Girder Deflections - Two Panels

Girder Deflections - Two Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0316	-0.0316	-0.0316	-0.0316	-0.0365	-0.0443	-0.0365	-0.0286
11.00	-0.0406	-0.0406	-0.0406	-0.0406	-0.0443	-0.0547	-0.0469	-0.0469
13.17	-0.0399	-0.0399	-0.0399	-0.0399	-0.0443	-0.0495	-0.0417	-0.0469
17.50	-0.0299	-0.0300	-0.0300	-0.0300	-0.0339	-0.0391	-0.0330	-0.0365

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 37: Trial 4 Girder Deflection Differences - Two Panels

Differences between SAP2000® and Lab Test Girder Deflection - Two Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0049	-0.0127	-0.0049	0.0029	13%	29%	13%	-10%
11.00	-0.0037	-0.0141	-0.0063	-0.0063	8%	26%	13%	13%
13.17	-0.0044	-0.0096	-0.0017	-0.0070	10%	19%	4%	15%
17.50	-0.0039	-0.0091	-0.0031	-0.0065	12%	23%	9%	18%

Stage 3: Three Panels

The leveling screws for the third panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '3 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 38: Trial 4 Girder Deflections - Three Panels

Girder Deflections - Three Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0480	-0.0480	-0.0480	-0.0480	-0.0573	-0.0599	-0.0573	-0.0547
11.00	-0.0639	-0.0639	-0.0639	-0.0639	-0.0677	-0.0781	-0.0677	-0.0703
13.17	-0.0641	-0.0641	-0.0641	-0.0641	-0.0677	-0.0729	-0.0677	-0.0677
17.50	-0.0497	-0.0497	-0.0497	-0.0497	-0.0521	-0.0547	-0.0521	-0.0547

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 39: Trial 4 Girder Deflection Differences - Three Panels

Differences between SAP2000® and Lab Test Girder Deflection - Three Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0093	-0.0119	-0.0093	-0.0067	16%	20%	16%	12%
11.00	-0.0038	-0.0142	-0.0038	-0.0064	6%	18%	6%	9%
13.17	-0.0036	-0.0088	-0.0036	-0.0036	5%	12%	5%	5%
17.50	-0.0024	-0.0050	-0.0024	-0.0050	5%	9%	5%	9%

Stage 4: Four Panels

The leveling screws for the fourth panel were set to the required length and the panel was placed on the girders. The torque of the leveling screws was adjusted such that an equal torque was applied to each screw. The girders were then surveyed for their elevations and the deflections were calculated. An analysis was run in SAP2000® using the '4 panels' load case. The deflection results were tabulated for the girders and exported to Microsoft Excel®.

Table 40: Trial 4 Girder Deflections - Four Panels

Girder Deflections - Four Panels								
Distance (ft.)	SAP2000® Deflections (ft.)				Lab Test Deflections (ft.)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0586	-0.0586	-0.0586	-0.0586	-0.0651	-0.0729	-0.0677	-0.0651
11.00	-0.0794	-0.0794	-0.0794	-0.0794	-0.0833	-0.0938	-0.0859	-0.0859
13.17	-0.0807	-0.0807	-0.0807	-0.0807	-0.0859	-0.0885	-0.0807	-0.0833
17.50	-0.0644	-0.0644	-0.0644	-0.0644	-0.0651	-0.0703	-0.0651	-0.0677

The deflections calculated using the survey data were then compared to the girder deflections from SAP2000®. The percent difference between the two deflections was then calculated.

Table 41: Trial 4 Girder Deflection Differences - Four Panels

Differences between SAP2000® and Lab Test Girder Deflection - Four Panels								
Distance (ft.)	Girder Deflection Difference (ft.)				Percent Difference (%)			
	Girder A	Girder B	Girder C	Girder D	Girder A	Girder B	Girder C	Girder D
6.67	-0.0065	-0.0143	-0.0091	-0.0065	10%	20%	13%	10%
11.00	-0.0039	-0.0143	-0.0065	-0.0065	5%	15%	8%	8%
13.17	-0.0053	-0.0079	-0.0001	-0.0027	6%	9%	0%	3%
17.50	-0.0008	-0.0060	-0.0007	-0.0033	1%	8%	1%	5%

Final Profile of Bridge Deck

After all four of the panels were set onto the girders, the top surface of the bridge deck was surveyed. This survey was completed to determine if the constructed profile of the bridge deck matched the desired profile. Figure 50 shows the profile of the bridge deck along girders A and D and the desired profile of -1.4%.

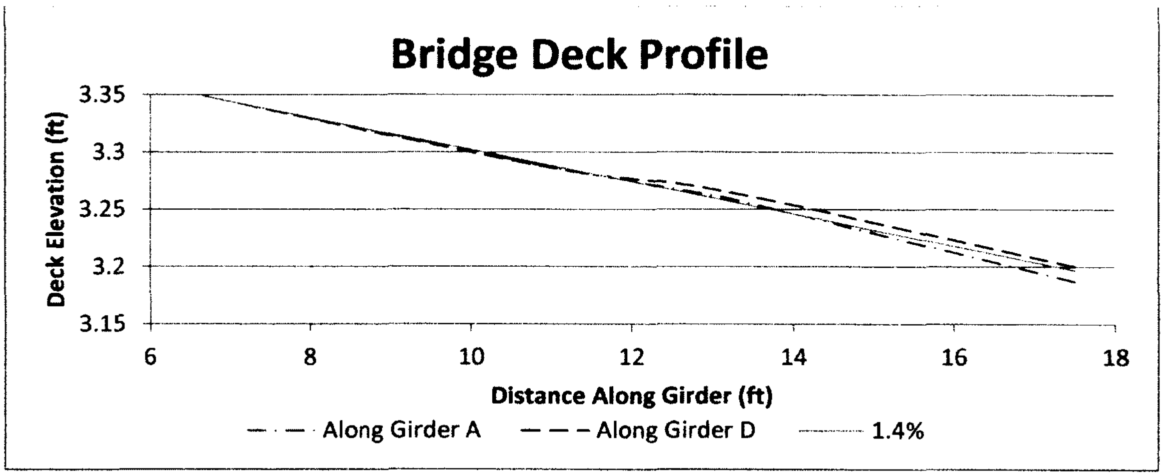

Figure 50: Trial 4 Bridge Deck Profile

The maximum difference between the actual profile and the desired profile of the bridge deck for trial 4 was 0.00306' or about 1/32nd of an inch.

3.6. Lab Trial Conclusions

The ability to accurately predict the deflections of the girders during the staged construction procedure became more apparent after each trial. The differences between the SAP2000® predicted deflections and the lab model observed deflections became less and less significant with each trial. This improved accuracy was accomplished by continuously critiquing and improving the methods used in both the prediction modeling and the lab procedures.

Accuracy in the modeling process was increased by using more precise methods of surveying, using a fourth order curve to represent the vertical profile of the unloaded girders, and determining the actual weight of the panels for use in the leveling screw load calculations.

The lab procedure accuracy was also improved. This was improved by using more precise surveying techniques as well as developing an effective technique for torqueing the leveling screws. Determining the relationship between leveling screw torque and axial load applied to the girders also aided in the equal distribution of the dead load of the panels to the girders.

After the completion of the final trial, the maximum difference in deflection between the predicted SAP2000® model and the observed deflection of the lab model was 0.014' or 0.168". This equates to a difference of less than three-sixteenths of an inch.

The results of the fourth trial show that the deflections in each of the girders of the lab model were greater than the predicted SAP200 girder deflections. This suggests that the girders used in the lab model do not behave identically to the frame elements modeled in SAP2000®. The fact that the lab model deflections were consistently greater than the SAP2000® model deflections provides reinforcement that the panel loads were distributed evenly across the girders and no single girder was overloaded with respect to an adjacent girder.

The ability to control the profile of the top surface of the completed bridge deck was also a success. The maximum difference in projected profile and observed profile in the fourth trial was 0.0031' or 0.037".

CHAPTER 4

4. TRANSVERSE JOINT MATERIAL TESTING

4.1. Need for Research

The cracking of the transverse joint, between panels, is the most common form of damage observed in full depth precast panel bridge decks. (Hieber, et al., 2005) These cracks allow for the infiltration of water and other chemicals which can begin to deteriorate the concrete and steel elements that make up the panel. This research aims to identify a structural adhesive, to be used in full depth precast panel bridge deck assembly, which will seal the transverse joint and prevent the cracking of the precast panels along the transverse joint.

During the assembly of the full width, prestressed deck panels, post tensioning bars will be installed in the longitudinal direction of the bridge, and stressed, to join the panels together and induce compressive forces in the concrete. The transverse joint, where the panels meet, will be subject to roughly 250 psi of compression once adjacent panels are fully post tensioned. Because the panels will not be "match cast" there is no guarantee that the profile of the transverse joint of a panel will match up perfectly with the panel it abuts. If this transverse joint is stressed in direct contact with another panel, some areas (high spots) of the joint will experience much higher stresses than the rest of the joint. This increased stress has the potential to cause some areas of the joint to crack and spall off, weakening the panel and creating an area that is vulnerable to chemical infiltration.

In order to evenly distribute stresses across the transverse joint, joining two panels that were not match cast, a structural sealant material is to be applied to the sides of the two adjoining panels to a thickness equal to or greater than half of the longitudinal panel length tolerance. This thickness is imperative to ensure that the entire length of the joint is sufficiently coated and that there is no direct concrete on concrete contact between the panels, when joined, causing stress concentrations.

Due to the complexity and accelerated timeline of the construction process proposed for pre-cast panel installation, the performance of the structural sealant material is very important. Some of the constraints of the material are as follows.

- Material must be resistant to chemical infiltration.
- Material properties must be equal to or greater than the ultimate strength and stiffness than the panel concrete properties.
- Physical properties of the material must not change dramatically due to change in temperature.
- Material must perform as intended in cold climates (sustained temperatures less than zero degrees Fahrenheit).
- Material must remain in a workable state for at least fifteen minutes after completion of mixing to allow workers to spread evenly along the entire length of the transverse joint of each panel.
- Material must bond to vertical surfaces of concrete and not slump.

- Material must not harden before the panels are joined together and joint is sealed.
- Material must harden to a strength of at least 250psi, one hour from the completion of mixing so that panels can be post tensioned without considerable loss due to creep.

Testing will be performed on multiple adhesives to determine the most suitable product given the constraints of its application.

4.2. Experimental Setup

To evaluate the physical characteristics of each material under load, an Instron Materials Testing Machine was used. Figure 51 shows the Instron Machine used for this research. The Instron Machine was programmed to apply a constant compressive force of 250psi on the sample regardless of the compressive displacement experienced during the test. The Instron Machine provided a data output of sample extension and force applied over the time allotted for the test. This data output could then be manipulated to determine the maximum and final compressive displacement of each sample over the period of time in which it was subject to the compressive load.

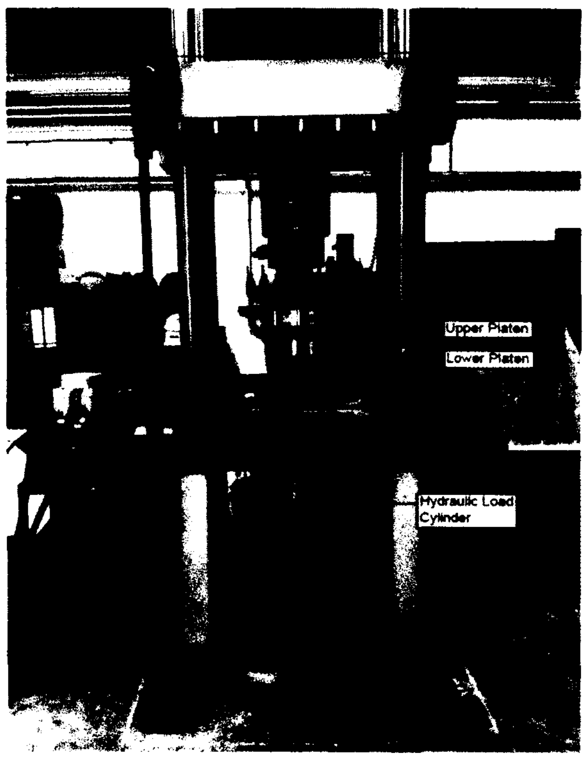

Figure 51: Instron Materials Testing Machine

The material sample to be tested in the Instron Machine was modeled to represent the conditions the material would be subject to during the construction process. A mold was built to produce a material sample 0'-6" long, 0'-6" wide and 0'-½" thick. Figure 52 shows a material sample in the mold. These dimensions were decided upon due to the size restraint of the Instron machine platens and the average predicted thickness of the material between the two transverse joints of the precast panels.

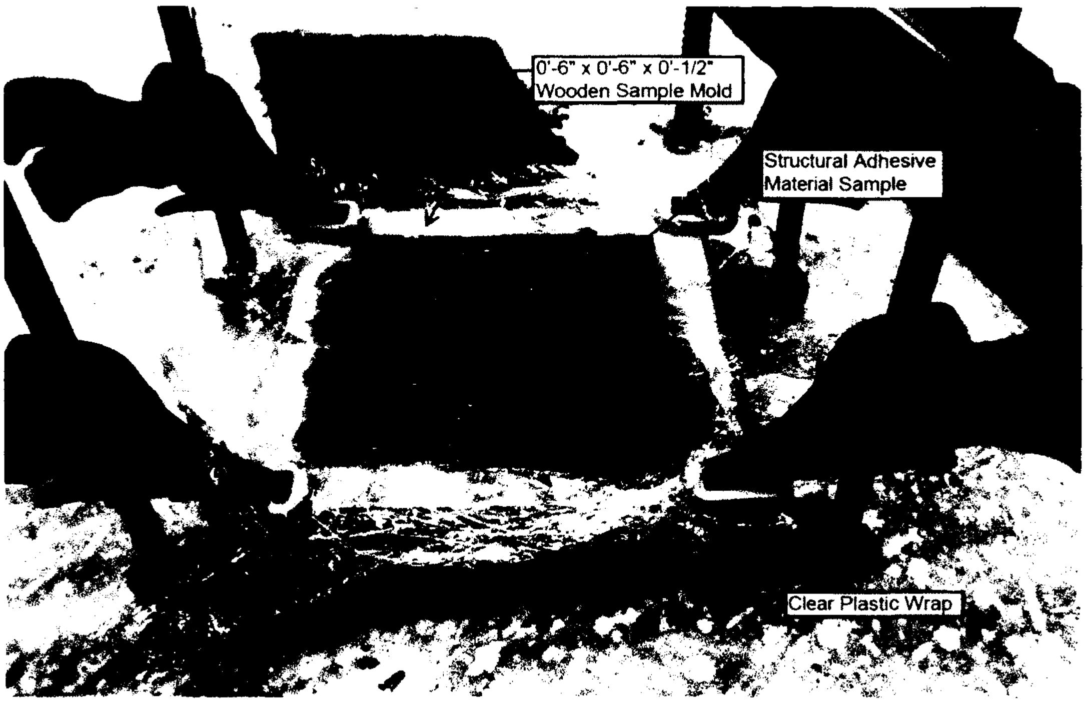

Figure 52: Emecole AB Material Sample in Mold

Three different materials were tested from two different companies using this method. These products were Sikadur 31, Hi-Mod Gel from Sika Corporation along with Emecole 455, Emecole A7400 B7654, Emecole AB, and Emecole CD from Emecole Incorporated. These five materials are classified as accelerated bridge construction adhesives. Each of these products is a two part mix that requires a specified blending ratio and process. Figure 53 shows the two parts to be mixed.

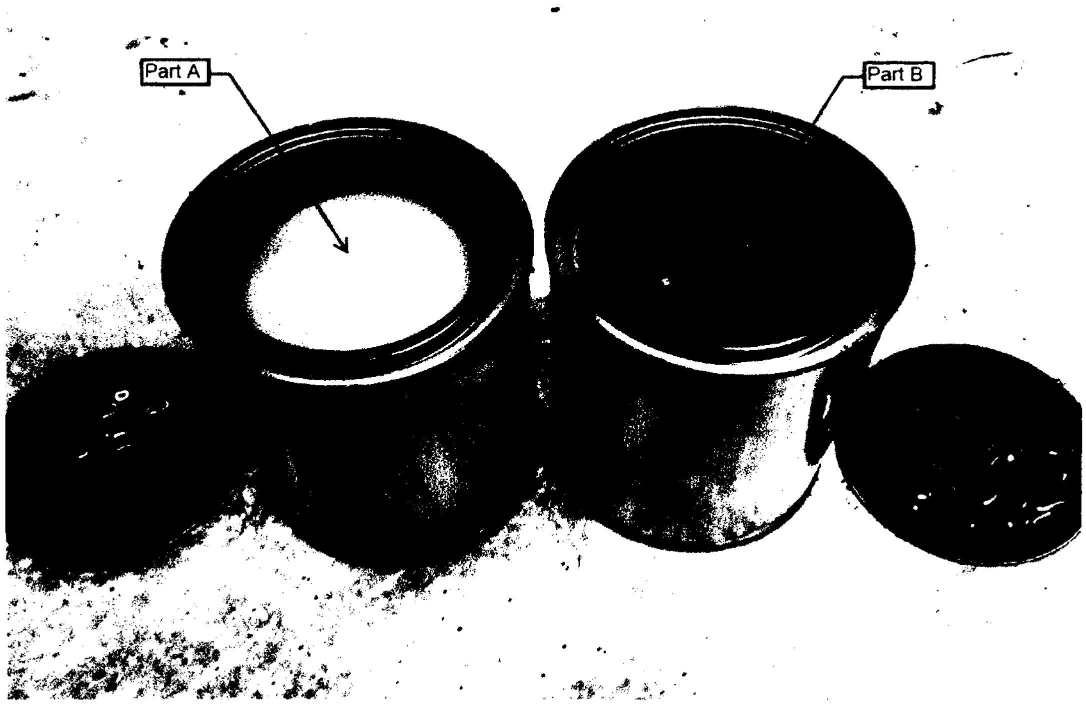

Figure 53: Emecole AB Part A and Part B

For each of these products, the same experimental process was followed. Each component, part A and part B, was measured by volume in accordance with the specified mix ratio. The two components were then mixed together according to manufacturer specifications. Once the two components were sufficiently mixed according to manufacturer specifications, the adhesive was placed in a 0'-6" x 0'-6" x 0'-½" mold and allowed to cure. While the sample was curing it was monitored for physical characteristics such as workability and pot life time. The sample was allowed to cure for a measured length of time or until it showed physical signs of being able to accept stresses of greater than 250psi without dramatic deflection due to loading. Once the sample was cured sufficiently, the sample was placed in the Instron Machine and the compression test initiated. The Instron Machine applied a constant load of 9,000 pounds or 250psi for the length of time specified for the test, generally four hours.

4.3. Testing Results

The five structural sealant materials were tested in the same fashion with varying cure times and Instron test times depending on the characteristics of the material. The testing dates and times are summarized in Table 42.

Table 42: Structural Material Sealant Test Information

Structural Sealant Material Test Information			
Product Name	Test Date	Sample Cure Time (hr.)	Instron Test Length (hr.)
Sikadur 31, Hi-Mod Gel	12/2/2011	4.0	2.0
Emecole 455	1/18/2012	1.0	4.0
Emecole 455	3/8/2012	1.0	4.0
Emecole A7400 B7654	4/11/2012	1.5	4.0
Emecole A7400 B7654	4/16/2012	2.0	4.0
Emecole AB	10/26/2012	1.0	4.0
Emecole CD	11/2/2012	1.0	4.0

All of the samples were tested in the Instron Machine with the same loading parameters and constant compressive load of 9000 lb. or 250 psi. The results of all of the tests are summarized in Table 43.

Table 43: Structural Sealant Material Test Results

Structural Sealant Material Test Results		
Product Name	Compressive Deflection (in)	
	Maximum	Final
Sikadur 31, Hi-Mod Gel	0.09381	0.08734
Emecole 455	0.07424	0.05325
Emecole 455	0.05342	0.03480
Emecole A7400 B7654	0.25407	0.23703
Emecole A7400 B7654	0.06756	0.04879
Emecole AB	0.13153	0.13145
Emecole CD	0.06888	0.06875

4.3.1. Sikadur 31, Hi-Mod Gel

Sikadur 31, Hi-Mod Gel, produced by the Sika Corporation, is a high modulus, high strength, structural, epoxy paste adhesive used in accelerated bridge construction when joining match cast elements together. Sikadur 31 is applied to the joining surface of the elements and acts as a lubricant and a sealant between the elements. Product information for Sikadur 31, Hi-Mod Gel can be seen in APPENDIX D.

On December 2, 2011 a test was performed on the Sikadur 31, Hi-Mod Gel to determine how it would perform as a structural sealant material. The two parts were mixed together per the manufacturer's instructions. Sika Corporation advises the use of a Sika Paddle and low speed drill for the mixing of the two components. The Sika Paddle is a helical shaped mixing paddle with a long shaft that attaches to an electric drill.

After the material was sufficiently mixed a sample was cast and left to cure. The remaining mixed material, not used for the sample, remained workable for nearly an hour after being mixed. The sample that was cast did not cure sufficiently enough to be tested until four hours after mixing.

After the four hour cure time, the material sample was placed in the Instron Machine and tested for two hours. The results of this test are displayed in Figure 54.

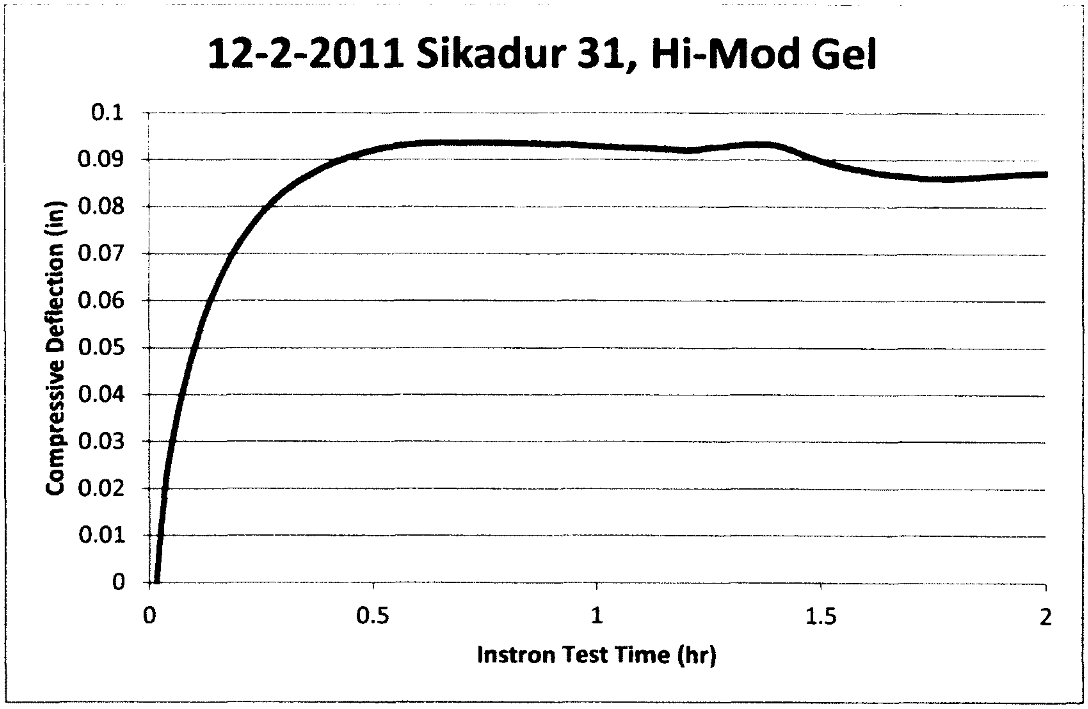

Figure 54: Sikadur 31, Hi-Mod Gel Test Results 12/2/2012

During the two hour Instron test the sample of Sikadur 31 experienced a maximum compressive deflection of 0.09381" and a final compressive deflection of 0.08734". Figure 55 shows the sample of Sikadur 31, Hi-Mod Gel after testing.

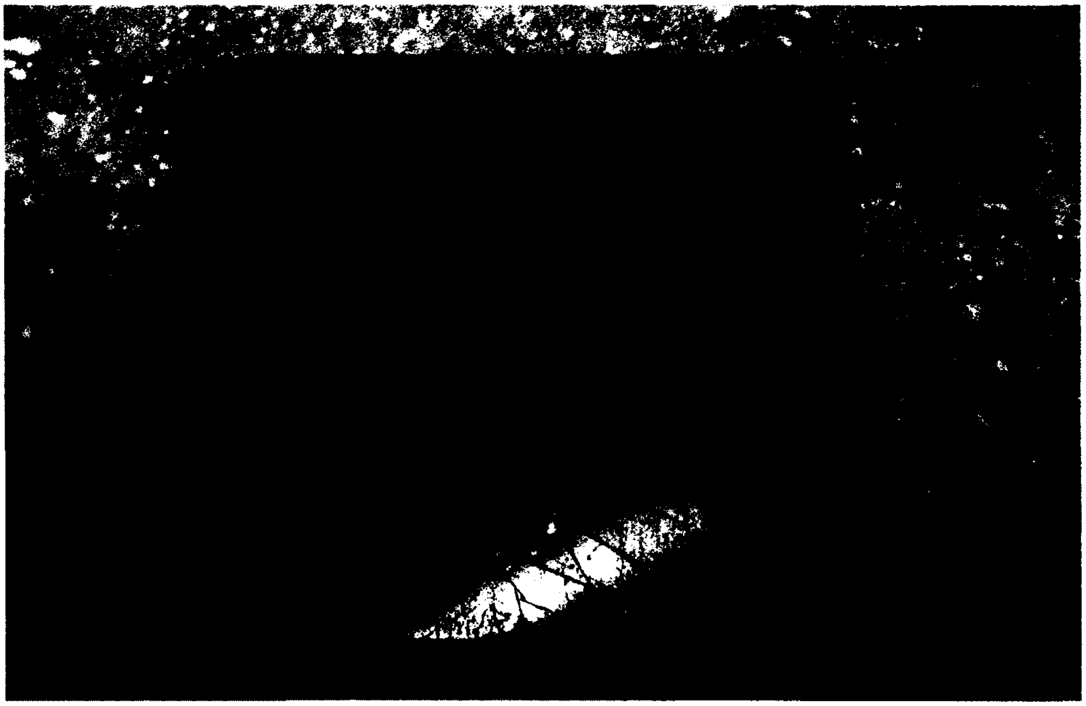

Figure 55: Sikadur 31, Hi-Mod Gel Sample

4.3.2. Emecole 455

Emecole 455 is a two-part, polymer based sealant produced by Emecole Inc. This material is generally used as a crack repair material for concrete. Product information for Emecole 455 can be seen in APPENDIX D.

On January 18, 2012 a test was performed to determine the suitability of the Emecole 455 sealant for use as a structural sealant material. After mixing the two parts for the recommended length of time, the sample was cast and left to cure. The remaining mixture, left over after the sample was cast, remained in a workable state for roughly ten to twelve minutes. This length of time was slightly less than what was specified for application to the transverse joints. After an hour of cure time the sample was placed in the Instron machine and tested. The results of the four hour test are displayed in Figure 56.

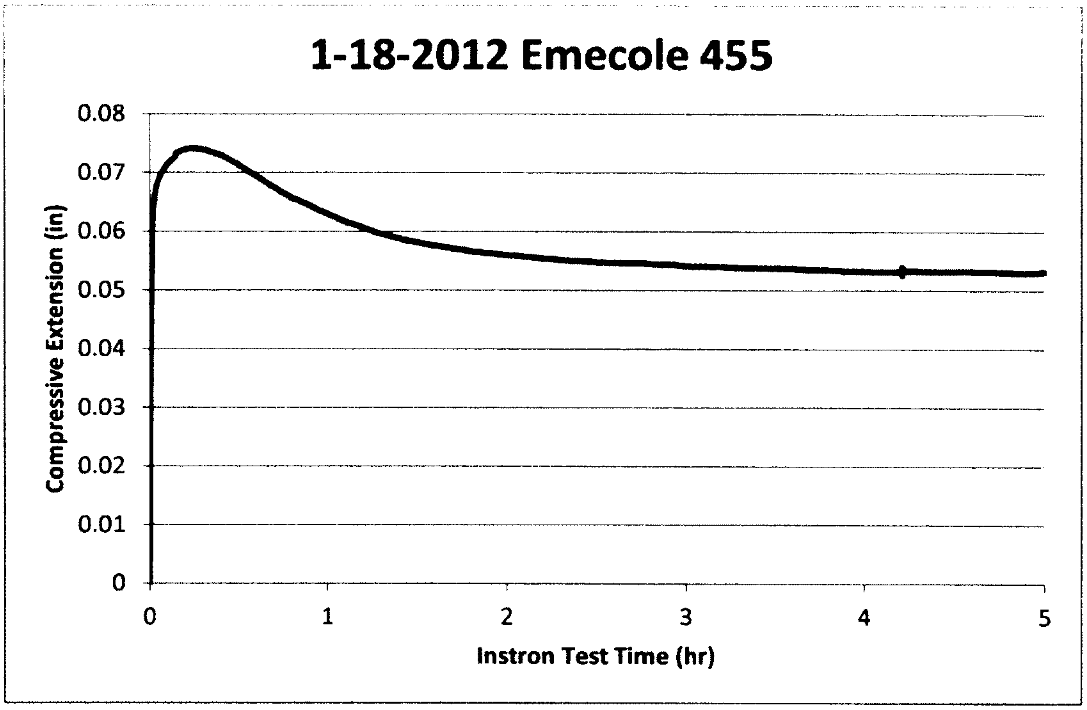

Figure 56: Emecole 455 Test Results 1/18/2012

The maximum compressive deflection of the sample was 0.07424" and occurred at roughly 0.25 hours after the start of the test. The final compressive deflection was 0.05325". Figure 57 shows a sample of Emecole 455 after testing.

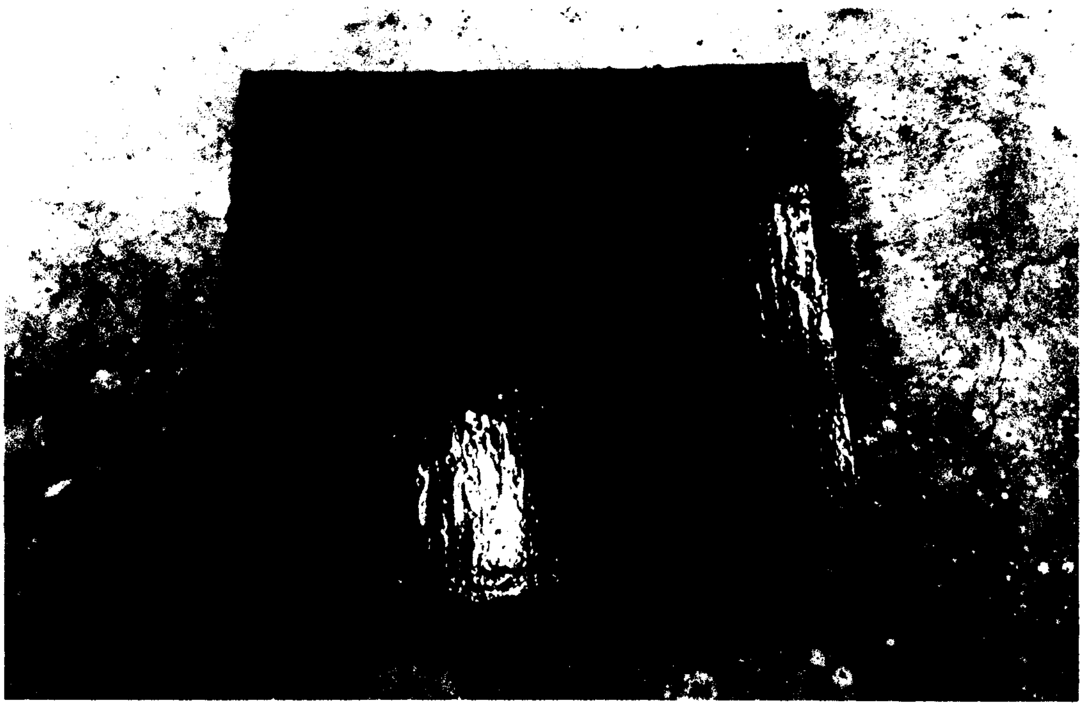

Figure 57: Emecole 455 Sample

On March 8, 2012 another test was performed on the Emecole 455 material. This test was done to attempt to replicate the promising results of the previous Emecole 455 test.

This test was done in the same fashion as the previous Emecole 455 test keeping constant the preparation procedures, material mixing, test parameters, cure time, and test length. The results of the test are displayed in Figure 58.

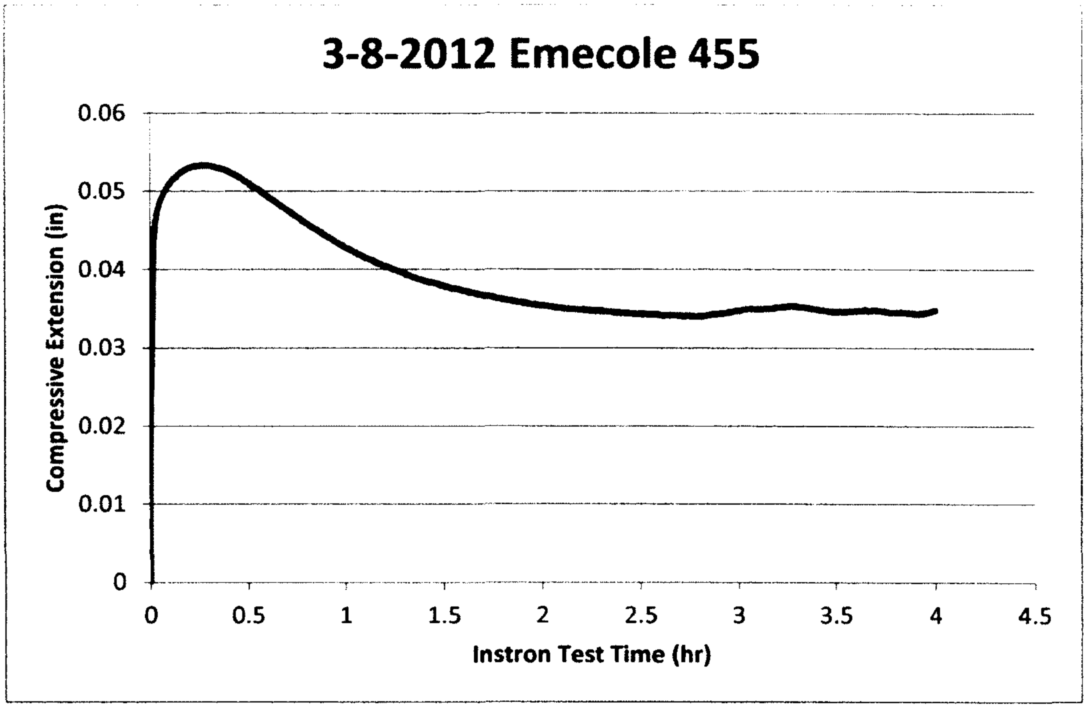

Figure 58: Emecole 455 Test Results 3/8/2012

The maximum compressive deflection of the sample was 0.05342" and occurred at roughly 0.25 hours after the start of the test. The final compressive deflection was 0.03480".

<u>4.3.3. Emecole A7400 B7654</u>

Emecole A7400 B7654, from Emecole Inc., is a polymer based structural sealant. This two part polymer is widely used for concrete crack repair.

On April 11, 2012 a test was completed to determine the performance of Emecole A7400 B7654 for use as a structural sealant material. After mixing the two parts for the recommended length of time, the sample was cast and left to cure. The remaining mixture, left over after sample was cast, remained in a workable state for roughly thirty minutes. This was a greater amount of time than what was required for application to the transverse joints. After an hour and a half of cure time the sample was placed in the Instron machine and tested. The results of the four hour test are displayed in Figure 59.

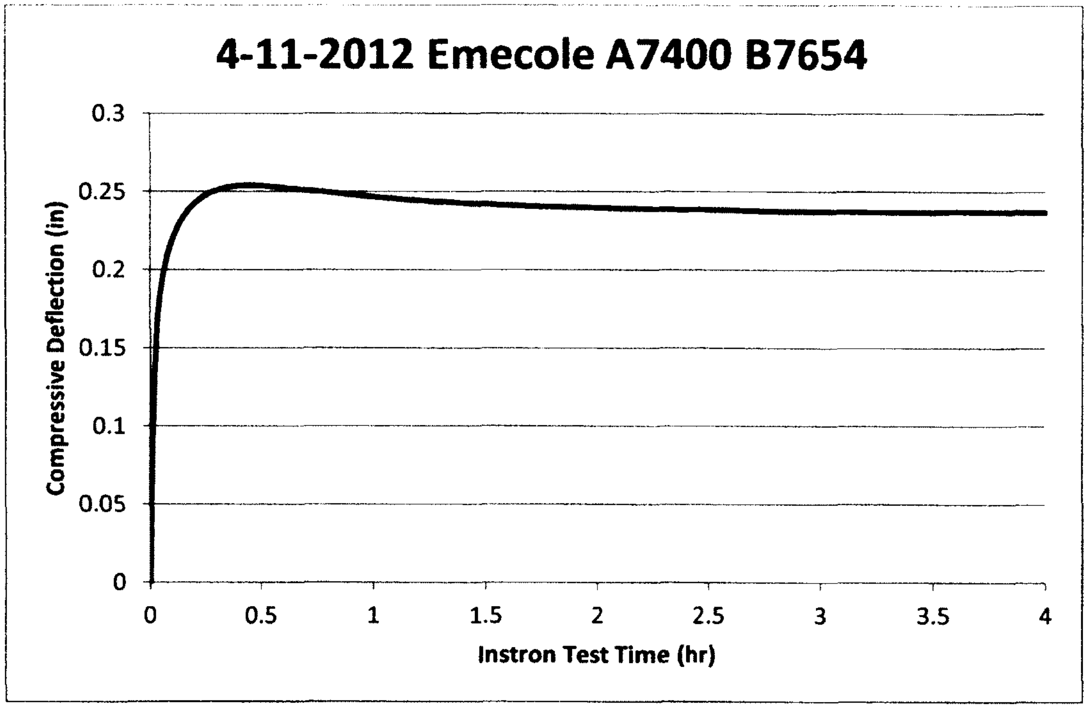

Figure 59: Emecole A7400 B7654 Test Results 4/11/2012

It is apparent by the results that the sample was not sufficiently cured when it was placed in the Instron Machine for testing. The original thickness of the sample was 0.5" and the results show a compressive deflection of the sample of 0.25407" at roughly 0.5 hours after the test began with a final compressive deflection of 0.23703". It is obvious that the sample required a longer cure time before it was tested. Figure 60 shows a sample of Emecole A7400 B7654 after testing.

Figure 60: Emecole A7400 B7654 Sample

On April 16, 2012 another test was performed on the Emecole A7400 B7654 structural sealant. The cure time of this test was lengthened, given the results of the previous test, to two hours. Both parts were mixed in the same fashion and a sample was cast. The extra material remained workable for roughly the same amount of time as in the first test, thirty minutes.

After two hours of cure time the sample was placed in the Instron machine and the compression test began. The results of the four hour test are displayed in Figure 61.

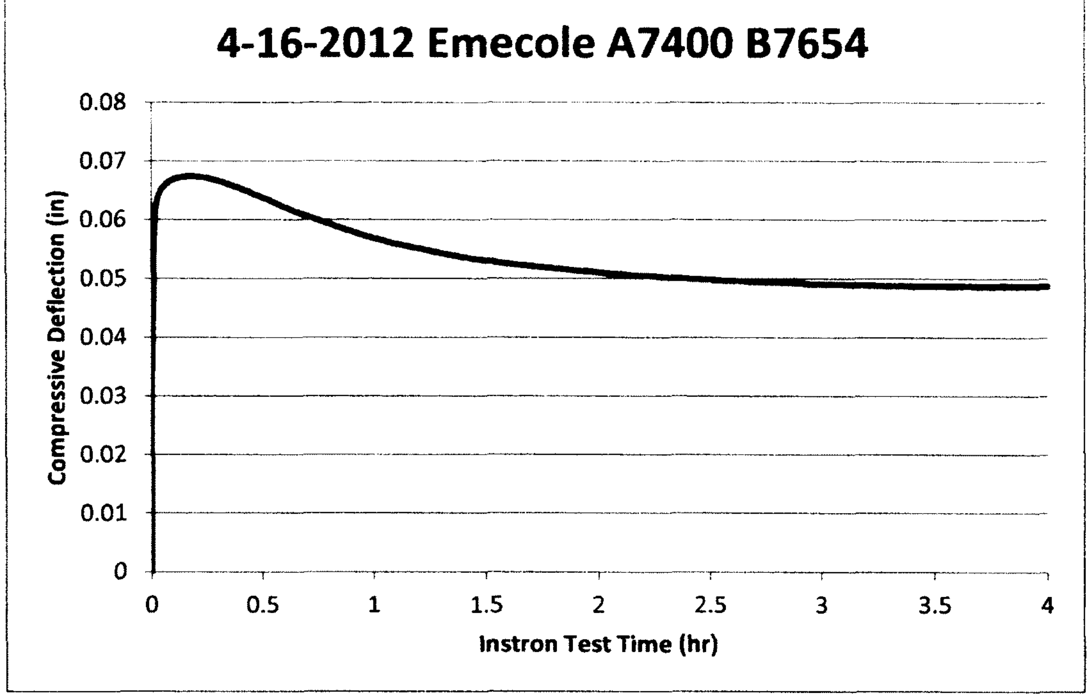

Figure 61: Emecole A7400 B7654 Test Results 4/16/2012

The results of this test were much more favorable than the first in terms of the compressive deflection experienced by the sample. The maximum compressive deflection was 0.6756" and occurred at a time of roughly 0.25 hours after the test began. The final compressive deflection of the sample was 0.04879".

4.3.4. Emecole AB

Emecole AB was a custom polymer based mix, designed by Emecole, Inc., which was developed to attempt to meet the specifications needed for the structural sealant material.

This material was tested in the same fashion as the other materials. After mixing the two parts for the recommended length of time, the sample was cast and left to cure.

The remaining mixture, left over after the sample was cast, remained in a workable state for roughly eight minutes. This length of time was considerably less than what was specified for application to the transverse joints. This test consisted of a one hour cure time and a four hour Instron test. The results of the Instron test are displayed in Figure 62.

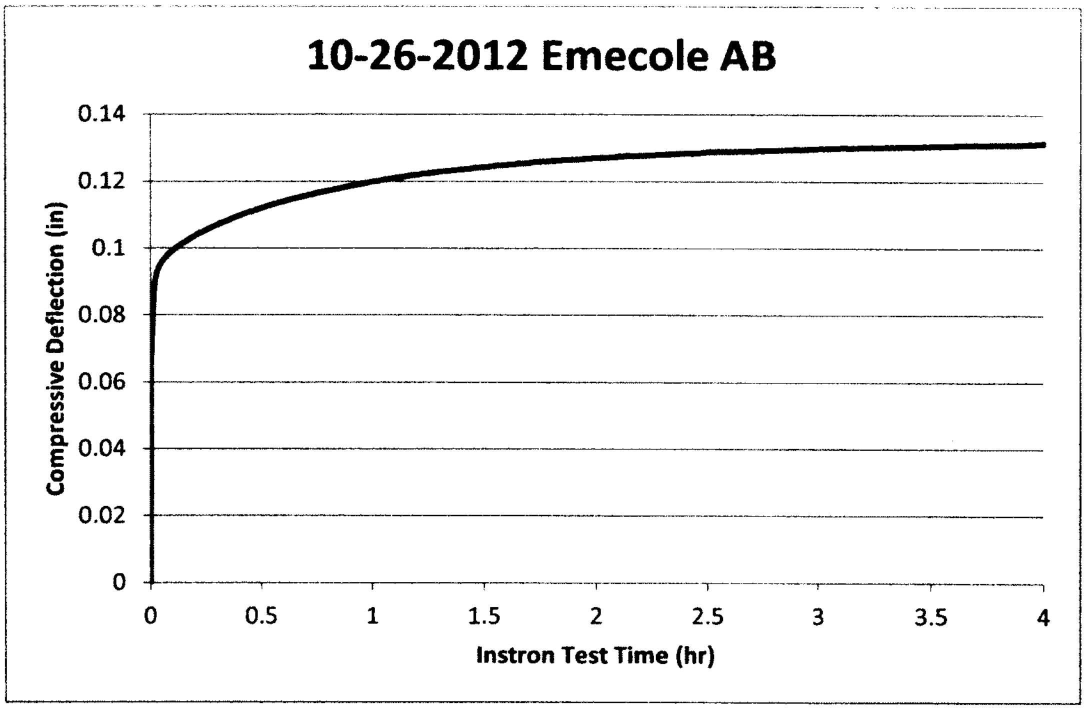

Figure 62: Emecole AB Test Results 10/26/2012

The maximum compressive deflection of the sample was 0.13153" and occurred at the very end of the test. The final compressive deflection was almost identical to the maximum compressive deflection of the sample and was 0.13145". Figure 63 shows a sample of Emecole AB after testing.

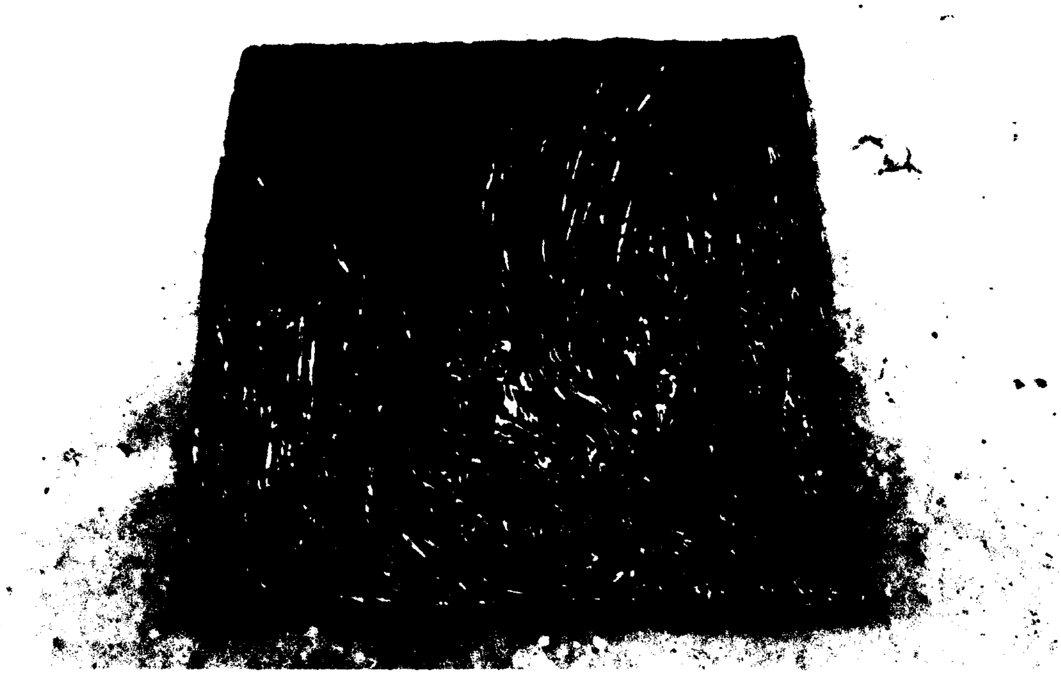

Figure 63: Emecole AB Sample

4.3.5. Emecole CD

Emecole AB was another custom polymer based mix, designed by Emecole, Inc., which was developed to attempt to meet the specifications needed for the structural sealant material. This material was made after the Emecole AB testing was finished. It was an attempt to refine the previous mix design.

This material was tested in the same fashion as the other materials. After mixing the two parts for the recommended length of time, the sample was cast and left to cure. The remaining mixture, left over after the sample was cast, remained in a workable state for roughly nine minutes. This length of time was less than what was specified for application to the transverse joints. This test consisted of a one hour cure time and a four hour Instron test. The results of the Instron test are displayed in Figure 64.

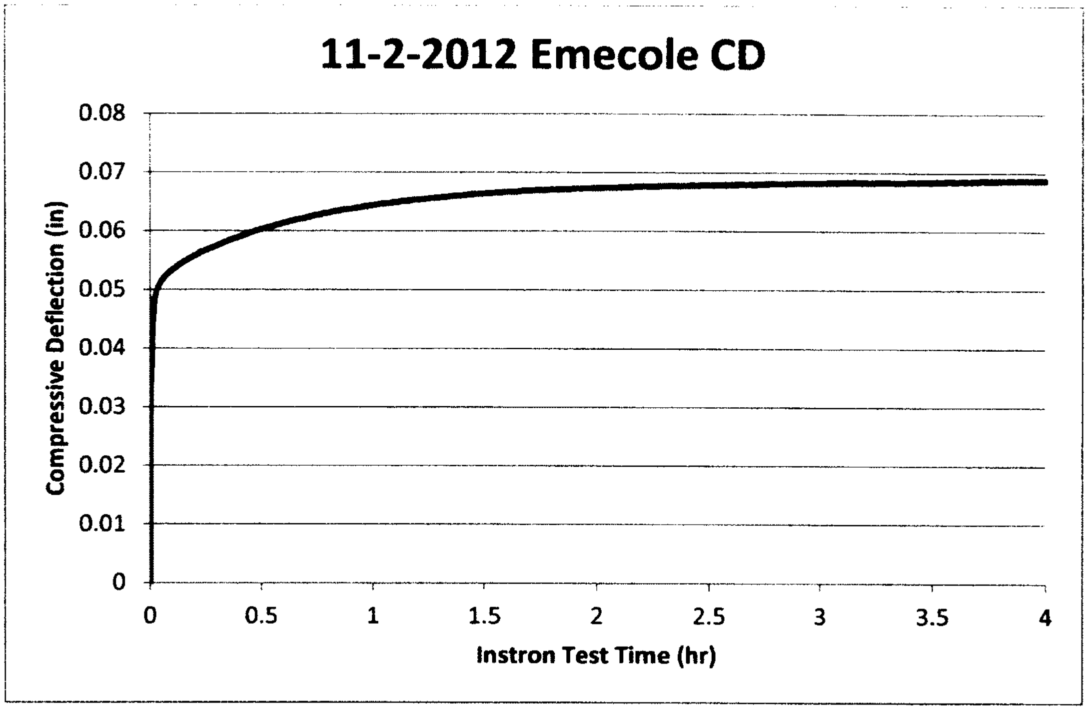

Figure 64: Emecole CD Test Results 11/2/2012

The maximum compressive deflection of the sample was 0.06888" and occurred at the very end of the test. The final compressive deflection was almost identical to the maximum compressive deflection of the sample and was 0.06875". Figure 65 shows a sample of Emecole CD after testing.

Figure 65: Emecole CD Sample

4.4. Testing Conclusions

After the testing of the five structural adhesives, conclusions were made, based on the constraints, as to which materials could perform well as a transverse joint material.

4.4.1. Sikadur 31, Hi-Mod Gel

After testing the Sikadur 31, Hi-Mod Gel it was clear that this sealant did not meet the required specifications of the structural sealant material needed for the transverse joints.

The pot life of the material was well beyond what was needed for application but the cure time was far too long. The material required a cure time of four hours before it was able to be tested. The compressive deflection of the sample during the testing was also higher than desired.

It is not recommended that more research be completed pertaining to Sikadur 31, Hi-Mod Gel be used as a structural sealant material.

4.4.2. Emecole 455

After testing the Emecole 455 material it was clear that this sealant could meet the required specifications of the structural sealant material needed for the transverse joints.

The pot life of this material was in the range of 10-12 minutes. This is less than the required workability time. It is possible that a retarding agent be used in the mix design to provide a longer workability time. The cure time of the sample was very desirable at one hour. After the one hour cure time, the sample performed well under load. The compressive deflection of the sample was lower than the majority of the samples tested.

It is recommended that Emecole 455 be researched further for use as a structural sealant material.

4.4.3. Emecole A7400 B7654

After testing the Emecole A7400 B7654 material it was clear that this sealant did not meet the required specifications of the structural sealant material needed for the transverse joints.

The pot life of the material was beyond what was needed for application but the cure time was too long. The material required a cure time of two hours before it was able to be tested. The compressive deflection of the sample during the test was within the same tolerances of the other materials once the sample was allowed to cure for two hours.

It is not recommended that more research be completed pertaining to Emecole A7400 B7654 being used as a structural sealant material.

4.4.4. Emecole AB

After testing the Emecole AB material it was clear that this sealant did not meet the required specifications of the structural sealant material.

The compressive deflection of this material was 0.13145 inches. This deflection was higher than other materials tested after the one hour cure time. This material would require greater than one hour of cure time to develop the strength needed to experience less compressive deflection. This material was also only workable for roughly eight minutes. This is well below the length of time needed for application to the transverse joint.

It is not recommended that more research be completed pertaining to Emecole AB being used as a structural sealant material.

4.4.5. Emecole CD

After testing the Emecole CD material it was clear that this material showed promising signs of meeting the required specifications of the structural sealant material.

The compressive deflection of this material was 0.06875 inches. This deflection was quite small compared to some of the other materials tested. This material was able to be tested after one hour of cure time which meets the specifications needed. One problem encountered with this material was that the workability time of the material was very short. The workability time for this material was less than ten minutes. This would not be enough time for the workers to apply the material to the transverse joint and join the two surfaces together. This workability time could possibly be increased by altering the mixture.

It is recommended that more research and mixture designs be completed pertaining to Emecole CD for use as a structural sealant material.

CHAPTER 5

5. CONCLUSIONS AND RECOMMENDATIONS

The results of this research indicated that this construction procedure could be used for the replacement of a bridge deck with full depth concrete panels. These research results also identified areas of potential error within the testing procedure which should be investigated before implementation of these methods and materials.

5.1. Leveling Device Settings Conclusions

The lab trial results conclude that the process outlined in this research was successful at calculating the length of each leveling screw in order to match the desired profile of a full depth precast panel bridge deck. The lab trial results also conclude that the torque applied to each leveling screw can be used to control the axial load each leveling screw applies to the girder. The lab trial results did however indicate that some error exists within the procedural methods used in this research.

5.1.1. Leveling Screw Length

The process outlined in this research has proven to be a viable method to calculate the required lengths of leveling screws for sloped full depth precast panel bridge decks. These leveling screw lengths were calculated and set prior to the placement of the full depth panels. The results of the survey of the top surface of the completed bridge deck for trial 4 confirmed that the calculation of the leveling screw lengths, prior to panel placement, was successful. The final profile of the completed bridge deck matched the desired profile to within 1/32nd of an inch which is less than the manufacturing tolerance of the precast panel.

Although the leveling screw lengths calculated produced a top surface profile almost identical to the desired profile there were sources of error identified within this calculation. The main source of error within the leveling screw length calculation was the deflections of the girders, with all panels applied, from the SAP2000® model. The actual deflections of the fully loaded girders were consistently greater than the deflections calculated by the SAP2000® model. This indicated that some error exists between the lab model and the frame element model in SAP2000®, and therefore also in the leveling screw length calculation. In order to correct this issue, a more accurate SAP2000® model must be built that better represents the conditions of the lab model.

It is important to note that, even with a deflection difference of 3/32nd of an inch between the lab model deflections and the SAP2000 girder deflections, the difference between the actual profile and desired profile of the bridge deck was 1/32nd of an inch. This indicated that a small error in the calculated deflections of the girders can exist and not drastically impact the final profile of the completed bridge deck.

5.1.2. Leveling Screw Torque

This research also showed that the adjustment of the torque applied to the leveling screws was successful at changing the axial load each leveling screw applied to the girder. The torqueing procedure and results outlined in this research showed the ability to correctly distribute the dead load of each precast panel to each girder. The actual deflections of the girders were consistently lower than the SAP2000 girder deflections for trial 4. These results were not optimal in terms of predicting deflections. The results did show that the girders were loaded uniformly with the exception of girder B, which was slightly lower than the rest.

This difference in deflection of one of the girders indicated that some degree of error existed within the torqueing procedure outlined in this research. This error could have been caused by the equipment used to torque the leveling screw or the condition of the leveling screw. Throughout this research, a torque wrench was used to torque the leveling screws to their required setting. The accuracy of this torque wrench was +/- 4%. This implied that an 8% difference in torque could have existed exist between leveling screws.

The condition of the leveling screws could also have impacted the torque settings. Prior to the lab trials within this research all of the leveling screws were removed, cleaned and greased. The grease on the leveling screws greatly affected the force required to spin them. If a leveling screw was not greased well it would require a greater amount of torque to turn than a well-greased leveling screw. This would allow the well-greased leveling screw to develop a higher axial load than a leveling screw that is not well-greased, for the same torque setting. A bent leveling screw will act in this same manner. If the leveling screw is slightly bent or warped it will require a greater amount of torque, than a straight leveling screw, to develop the same axial load.

5.2. Leveling Device Settings Recommendations

Although successful lab trials were conducted using the methods outlined in this research, there still exist many areas within this procedure in which error can be eliminated. Recommendations for improvements to this method are as follows.

5.2.1. Frame Element Model

The results of the final lab trial showed a consistently higher deflection in the girders compared to the calculated deflections of the structural analysis model. This suggests that discrepancies between the physical model and the structural analysis model exist.

One possible source of error within the lab model is the frame element support location. SAP2000® frame elements are modeled as a straight line between two joints. This frame element, modeled as a line and not a 3D object, has all of the physical section properties of the member it is representing. Because frame elements are not modeled as 3D elements, SAP2000® positions the joints that connect the frame elements at the neutral axis of the section. The location of the joint at the neutral axis creates discrepancies between the structural analysis model and the physical layout of the lab model.

The support conditions of the girders were modeled as joint restraints in the SAP2000® model. These restraints occurred at the location of the joint, which was located at the neutral axis of the girder. Figure 66 shows the support conditions modeled as joint restraints located at the neutral axis of the girders.

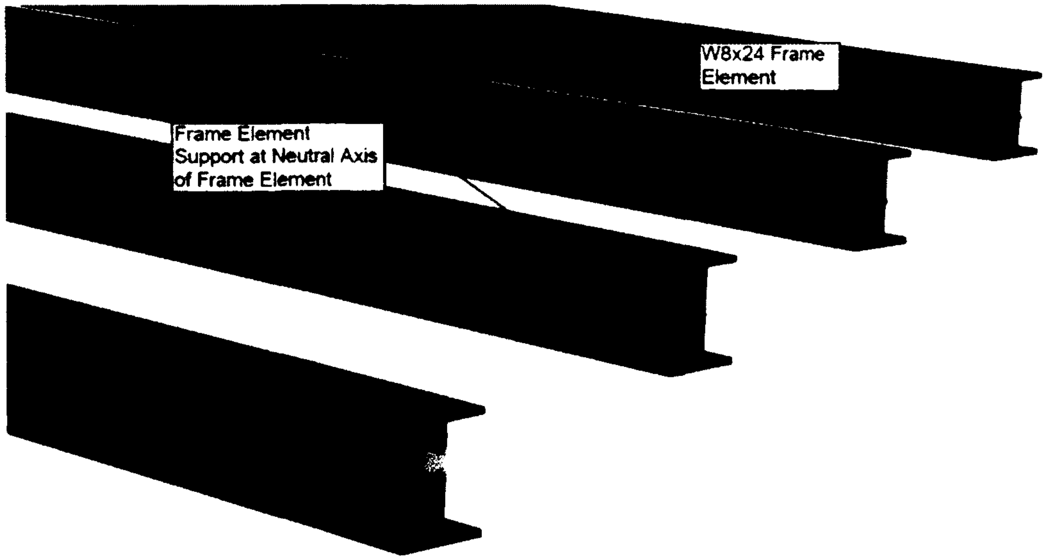

Figure 66: Frame Element Support Location

In the lab model, the girders rested on top of the rounded bearing plates. These bearing plates acted as the supports and were located at the bottom of the girder. Figure 67 shows the location of the rounded bearing plate, acting as the support for the lab model girder.

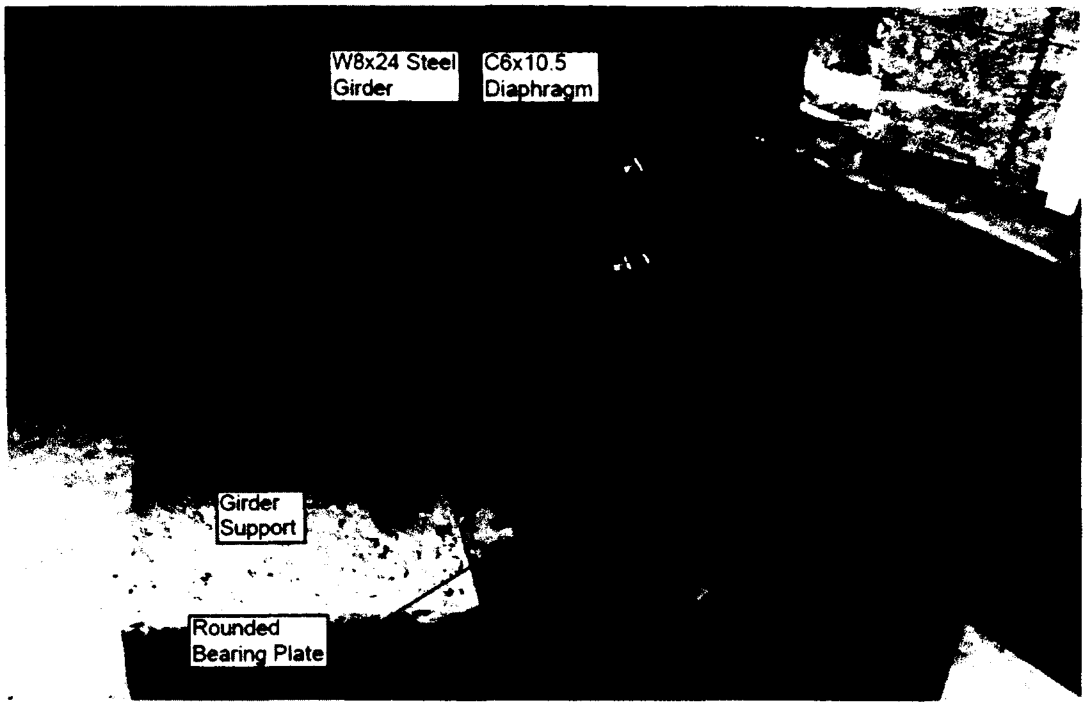

Figure 67: Lab Model Support Location

The distance between the physical location of the girder supports and the location in which they were modeled is equal to half of the girder depth or 0'-3 15/16". This difference could cause the deflections of the girders in the structural analysis model to be different than the actual deflections.

This same issue occurred at the locations of axial loading. The axial loads of the leveling screws were modeled as joint loads in the structural analysis model. The joints to which the load was applied are located at the neutral axis of the frame element. Figure 68 shows the 740.63 lb. joint load applied in the negative Z direction at the neutral axis of the frame element.

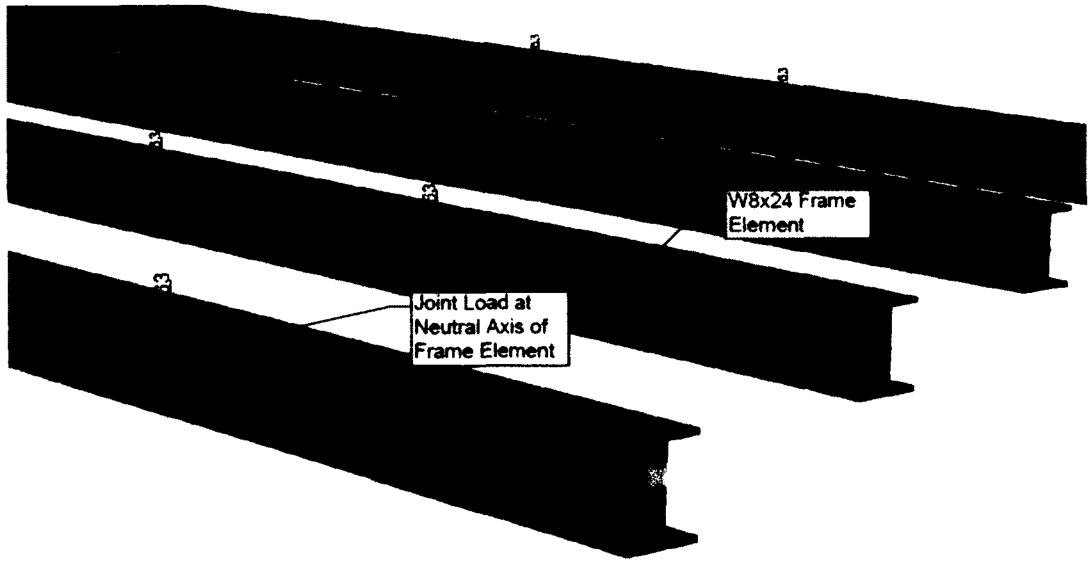

Figure 68: Frame Element Joint Load Location

The leveling screws of the lab model, that apply axial load to the girders, bear on the top flange of the girders. Figure 69 shows the leveling screws bearing on the top flange of the girder.

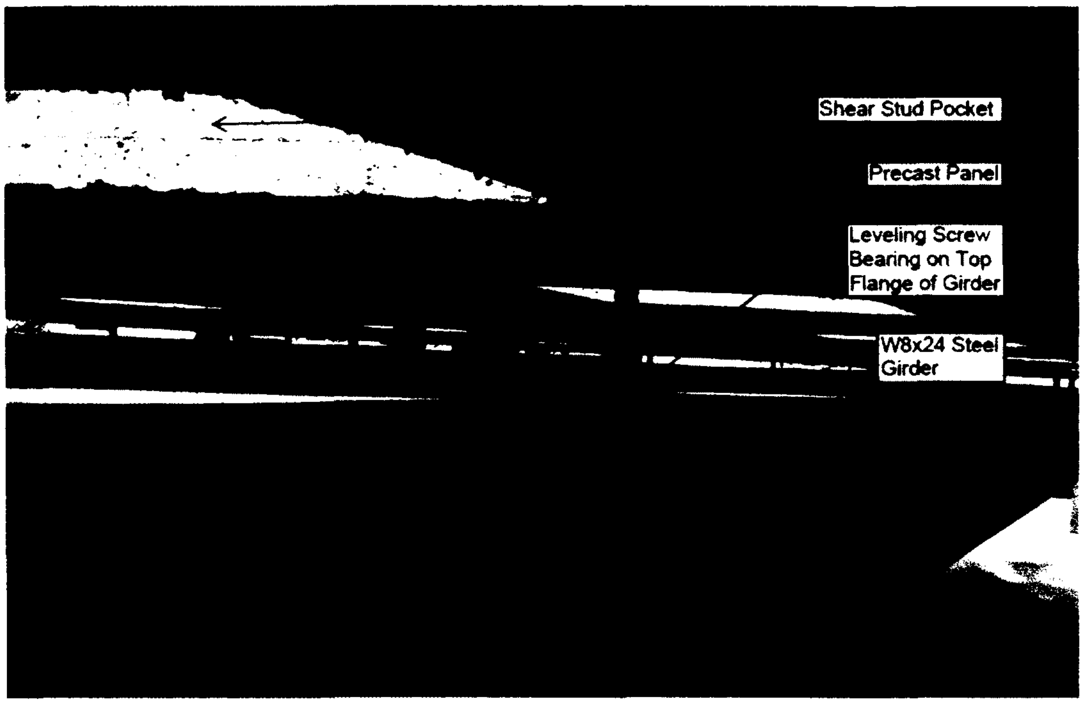

Figure 69: Leveling Screws Bearing on Girder

The assumption made by SAP2000® that all reactions occur at the neutral axis of the frame element could be a source of error in the deflections of the girders in the computer model. It is recommended that a parametric study within SAP2000® be completed to determine if any error is induced in the model due to the position of the frame element supports and axial loads at the neutral axis of the girder.

5.2.2. Girder Load Test

The results of the fourth lab trial showed that the actual deflections of the girders were consistently greater than the calculated deflections. These results could be caused by errors within the frame elements model, as described in section 5.2.1, but could also be caused by the difference in the physical properties of the girders in the lab and the properties assigned to the frame elements within SAP2000®. It is recommended that research be conducted to compare the bending properties of each of the girders to the bending properties of a frame element in SAP2000®.

5.2.3. Leveling Screw Torque Study

Throughout the lab trials expressed in this research, the axial load the leveling screw applied to the girder was altered by the changing the amount of torque applied to the leveling screw. To determine the relationship between the torque applied to the leveling screw, and the axial load it applied to the girder, a study was completed. (Section 3.2.6) It was discovered that a linear relationship existed between the torque applied to the leveling screw and the resulting axial load transferred to the girder.

This study was completed using a torque wrench with an accuracy of +/- 4%. The range of torque values tested within this study was from 25 in.-lb. to 190 in.-lb. This equates to an accuracy of +/- 1.0 in.-lb. at the low end of the test range and +/- 7.6 in.-lb. at the high end of the test range. The accuracy of the torque wrench used in this study could affect the results found.

The load cell used in this study could also be another source of error. The load cell readings were averaged after each leveling screw torque increase and read to the nearest 5 lb.

It is recommended that further research be conducted on the sensitivity of the torque applied to the leveling screws and its effects on the loading of the girders. The ability to correct the situation of an over loaded or under loaded girder, with the leveling screw settings of the succeeding panel, is important to reducing panel placement time. If an unacceptable difference in deflection is experienced, measures should be taken to correct these discrepancies in the settings of the next panel. This could possibly be accomplished by altering the distribution of the dead load of the next panel. The dead load applied to a particular girder can be increased or decreased by adjusting the leveling screw torque settings of the next panel to be placed. Altering the torque settings of the next panel can produce a higher or lower deflection in a particular girder.

5.2.4. Survey Procedure

The survey procedure used in the lab trials, to determine the deflection of the girders, was accurate to the nearest 1/32nd of an inch. Each girder was surveyed for elevation at 6 locations along its length using an Auto Level. None of these locations was located at the direct mid-span of the girder. Because of this, the maximum deflection, which would occur at the mid-span of the girders due to the loading configuration, of the fully loaded girders was not directly surveyed. It is recommended that for future lab trials, utilizing the laboratory setup identified in this research, an LVDT (Linear Variable Differential Transformer) be positioned at the mid-span of each of the girders to directly measure their maximum deflection.

5.2.5. Polynomial Curve

This research used the equation of a fourth order polynomial trend line, fit to the six survey elevations, to calculate the elevation of the top surface of the girder along its length. The fourth order curve was used because it produced a high coefficient of determination and did not appear to exaggerate the curvature of the girders. A fifth order polynomial curve was also fit to the same elevation data and produced a coefficient of determination of 1.0001 but appeared to slightly exaggerate the curvature of the girder. Because of this the fifth order curve was not used to model the curvature of the girders.

It is recommended that a study be completed in which a girder is surveyed for its elevation at six survey locations as well as at the midpoints between the six locations. It is recommended that a series of polynomial trend lines be fit to the elevations of the six survey locations. The equations of the polynomial trend lines should then be used to calculate the elevations of the midpoints between the survey locations. These elevations should then be compared to the surveyed midpoint elevations to determine the polynomial trend line that best represents the curvature of the girders.

5.2.6. Scalability

The construction procedures outlined in this research were performed on a single span bridge model measuring 24'-0" C-C of supports with an overall deck width of 16'-0". This procedure aims to be implemented on single span bridges ranging in lengths from 50'0" to 125'-0". It is recommended that a scalability study be performed to identify any problems related to this procedure being performed at a much larger scale.

5.2.7. Panel Weight

Throughout the lab trials performed in this research, it was assumed that all of the individual panel weights were equal. This assumption was made to simplify the modeling process of the lab system as well as to determine if a small variation in the weight between panels (5-10%) would affect the deflection of the girders. The results of trial 4 show that the I-End of all four girders was experiencing a consistently greater deflection than the J-End of the girders. This suggested that the panels on the I-End of the girders were applying more dead load than those on the J-End. It is recommended, due to these results, that the actual and not the average weight of the panels be used for the girder deflection analysis.

To successfully accomplish this, it is suggested that a more accurate weight be achieved for each panel. The study that was done as part of this research, Section 3.2.5, in which all of the panels were weighted using wheel load scales, weighed the panels at fifty pound increments. This fifty pound accuracy was the highest achievable with the use of the wheel load scales. It is recommended that the panels be weighed individually, using a tension load cell, and these weights be used in the SAP2000® deflection analysis.

5.2.8. Panel Stiffness

It is also recommended that the effect of the stiffness of the precast concrete panels be taken into account in the modeling process. This should be done to more accurately predict the required length setting of each leveling screw. It is assumed that the stiffness of the precast panels will affect the axial loads applied to the girders by the leveling screws as the panels are set and post tensioned together. Once two or more panels are post tensioned together they will no longer act independently of each other. These panels will begin to take on the flexural characteristics of a large plate. It is important to conduct further research to identify these effects.

5.2.9. Model Updating and Analysis Procedure

The model updating and analysis process as described in this research utilized SAP2000® and Microsoft Excel®. Survey data was collected during the trials and input into Excel®. This data was then converted to the correct format and input into SAP2000® to update the frame element model. The frame element model was then analyzed to determine the deflection of each girder for each panel loading sequence. This deflection data was then exported back to Excel® to calculate the leveling screw length and compare SAP2000® deflections to lab trial deflections. The process of transferring data back and forth between SAP2000® and Excel® was tedious and time consuming.

This analysis process is expected to be completed in real time while the bridge deck construction is ongoing. It is necessary, for this analysis process to be a viable option for use in construction, that the process of data transfer and analysis be streamlined. It is recommended that an API (Advanced Programming Interface) be developed and utilized to reduce the time necessary to update the SAP2000 frame element model with survey data, run analysis to calculate deflections of the girders, and then calculate leveling screw lengths.

5.3.Transverse Joint Material Testing Conclusions

The transverse joint material testing performed as a part of this research identified two materials that conformed to the constraints as outlined in Section 4.1. These two materials were Emecole 455 and Emecole CD

The Emecole 455 material average compressive deflection during the four hour test was 0.044" or 8.805%. The Emecole CD compressive deflection during the four hour test was 0.06875" or 13.75%. The Emecole 455 material yielded a lower deflection than the Emecole CD material when tested. This smaller deflection of the material is desired.

The transverse joint material will be applied to the transverse joint of each panel before they are joined. After the panels are joined and the material has cured the panels will be post tensioned together. The forces due to post tensioning will be roughly 250 psi in compression and be applied directly to the transverse joint material. Ideally the deflection in this material subject to a 250 psi compressive load would be minimal. The compressive deflection of the transverse joint material will affect the tension in the post tensioning bars. If the transverse joint material deflects too much, all tension will be lost in the post tensioning bars.

Emecole 455 and Emecole CD were both able to support 250 psi in compression one hour after being mixed without considerable compressive deflection but the workability time of the material was shorter than what was desired. A fifteen minute workability time was desired to allow for the mixing of the material, the application of the material to the transverse joint of the panels, and the joining of the panels. Any shorter period of time could create issues within the construction sequence including the curing of the sealant before the panels are joined. This scenario would prevent a good seal within the transverse joint of the panels.

5.4. Transverse Joint Material Testing Recommendations

It is recommended that additional transverse joint polymer testing be completed. Recommendations for additional testing and procedures are as follows.

5.4.1. Environmental Effects

The testing that has been completed was conducted strictly in laboratory conditions. It should be noted that even though the manufacturer of the material does not predict any changes in material properties or cure characteristics due to a difference in temperature, the effects should be investigated.

It is recommended that a series of tests be completed in which the material is subject to a range of temperatures during the curing process. These tests will show whether hot or cold temperatures accelerate or retard the cure time of the sample.

5.4.2. Alternative Materials

It is recommended that a wider range of structural adhesives be tested. Out of the five adhesives tested it was very clear which product performed the best given the conditions of the construction process. It is also very possible that another adhesive mixture exists, or could be created, that would perform better than those that were tested within this research.

5.4.3. Post Tensioning Losses

It is recommended that a study be conducted on the post tensioning losses due to initial deflection of the transverse joint adhesive as well as the long term defection of the material due to creep. If the stress losses in the post tensioning bars is significant, due to the deformation of the transverse joint material, initial overstressing of the post tensioning bars would be necessary to ensure the proper compressive force exists in the `completed bridge deck.

5.4.4. Sample Dimensions

The sample dimensions of the structural adhesive material tested as a part of this research (Chapter 4) were 0'-6" x 0'-6" x 0'-1/2". This sample size was tested because it yielded the largest possible sample area and conformed to the expected material application thickness. The sample area was limited by the platen size of the Instron machine used during the testing. The expected application thickness of the sample was 0'-1/2" which is half the dimension of the longitudinal length tolerance of the precast panels. The material sample size tested was very specific to the transverse joint application and did not conform to other standard creep tests. It is recommended that further research be conducted on the creep behavior of structural adhesives and standard creep test procedures be investigated.

5.4.5. Application to Transverse Joint

The scope of this structural adhesive material testing did not include the application of the material to the angular tongue and groove surface of the transverse joint. It is recommended that research be conducted on the application of the structural adhesive material to the angular tongue and groove joint and the joining of the two surfaces. It is also recommended that research be conducted on the performance of the structural adhesive material during the post tensioning process.

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APPENDICES

APPENDIX A

List of Terms

Frame Element: An object used to model beams, columns, braces, and trusses

Leveling Screw: A device used to position precast elements such that the desired profile of the structure is achieved. Also used to transfer the dead load of the precast element to the structure it bears on.

Load Pattern: A spatial distribution of loads upon a structure

Load Case: How loads will be applied to the structure and how the response of the structure is to be calculated

Joint: An object within SAP2000® that allows for the connection of elements, the application of loads and the location of results

Post tensioning: A process in which compression is induced in cured concrete by the stressing of reinforcement

Pre Stressing: A process in which the reinforcement is stressed in a casting bed before the concrete is cast. After the concrete has set the reinforcement is cut causing the tension in the reinforcement to induce compression in the concrete

SAP2000®: A structural analysis software package produced by Computers and Structures, Inc.

Solid Element: An object used to model three-dimensional solids.

Transverse Joint: The joint, between two full depth precast panels, that runs perpendicular to the length of the bridge

APPENDIX B

Calculations

Calculated Weight of Panels

$$W_{panel} = \gamma_{concrete} V_{panel}$$

$$\gamma_{concrete} = 150\ pcf$$

$$V_{panel} = 44.8\ ft^3$$

$$W_{panel} = (150\ pcf)(44.8\ ft^3)$$

$$W_{panel} = 6720\ lb$$

Trial 1 Leveling Screw Load

$$w = \frac{W_{panel}}{n}$$

$$w = weight\ per\ leveling\ screw$$

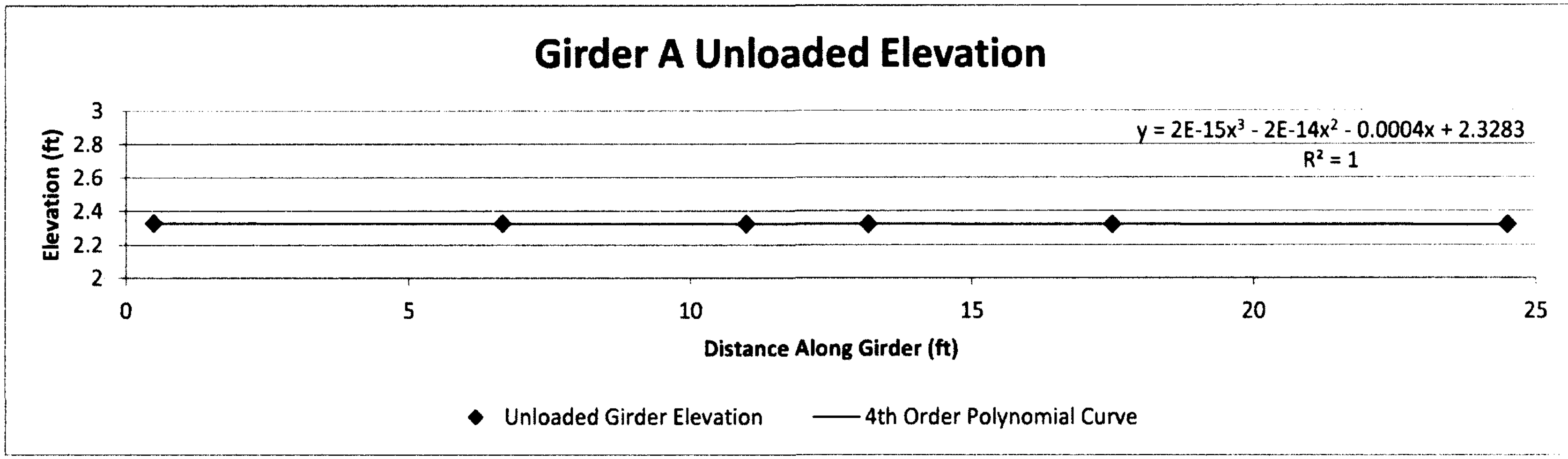
$$n = 8\ leveling\ screws\ per\ panel$$

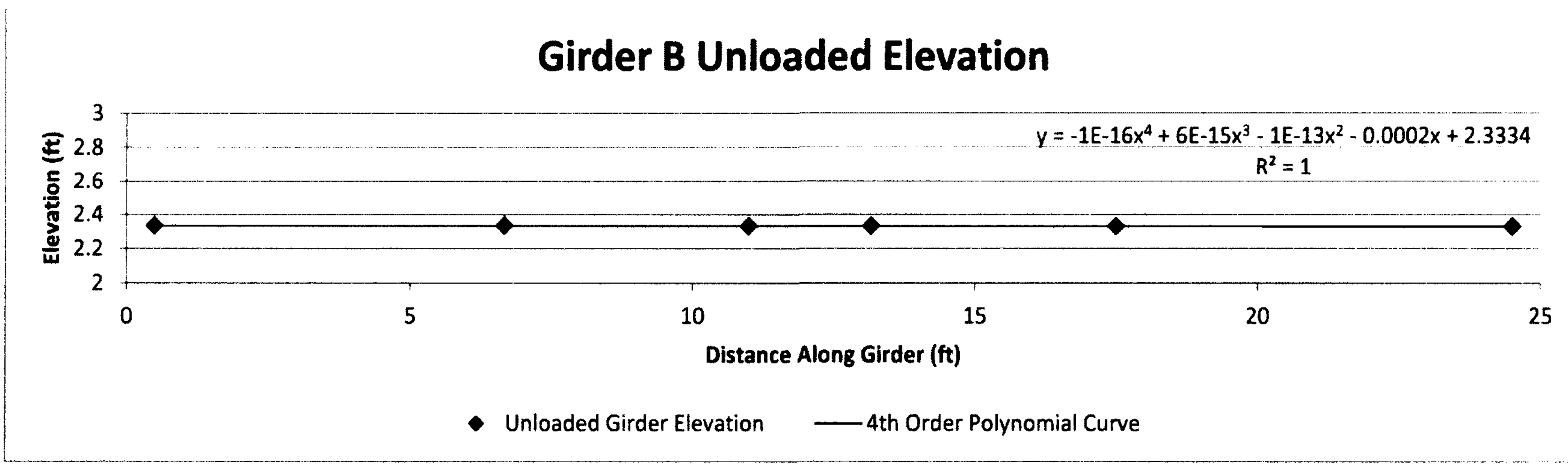
$$w = \frac{6720\ lb}{8}$$

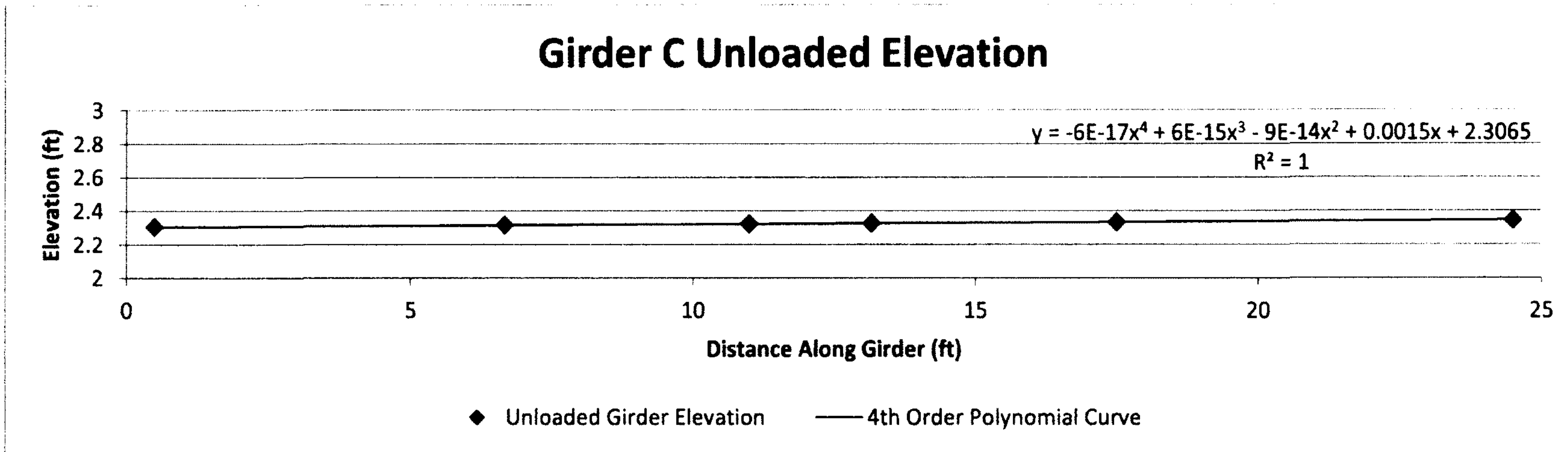
$$w = 840\ lb$$

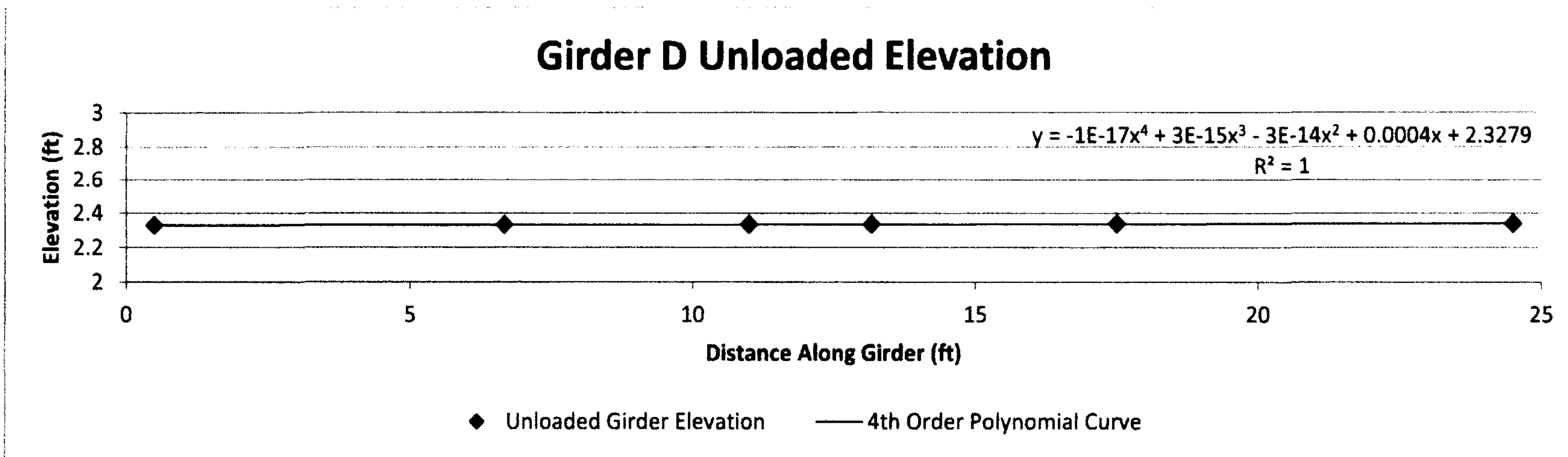
APPENDIX C

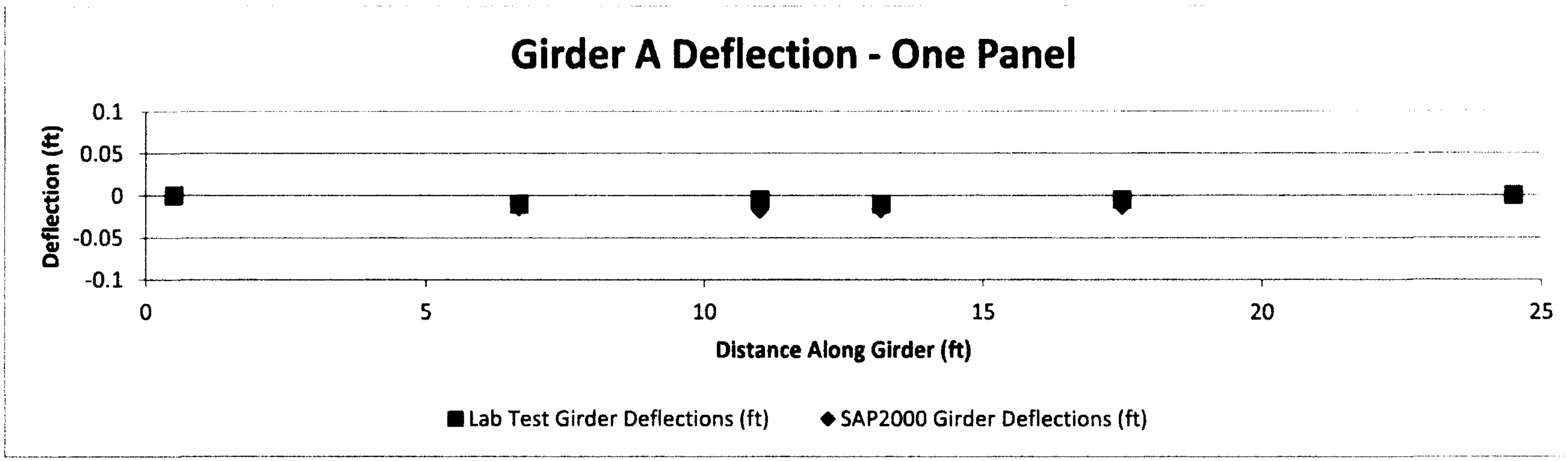
Trial 1 Girder Deflection Graphs

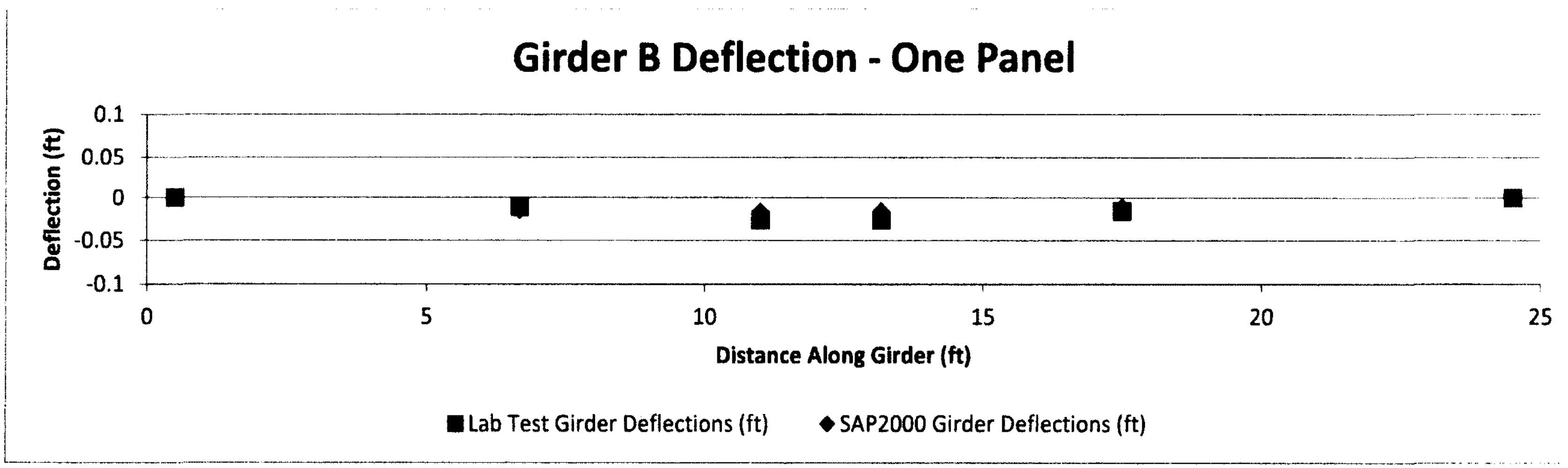


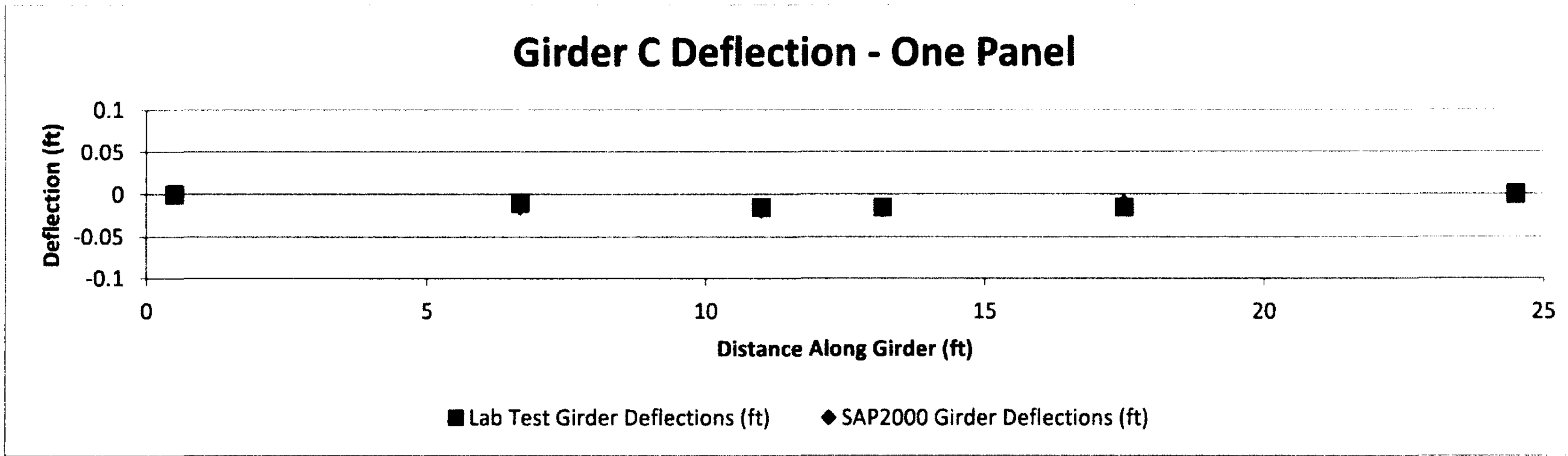



Girder C Unloaded Elevation
$y = -6E-17x^4 + 6E-15x^3 - 9E-14x^2 + 0.0015x + 2.3065$
$R^2 = 1$
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Unloaded Girder Elevation
4th Order Polynomial Curve


Girder D Unloaded Elevation
$y = -1E-17x^4 + 3E-15x^3 - 3E-14x^2 + 0.0004x + 2.3279$
$R^2 = 1$
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Unloaded Girder Elevation
4th Order Polynomial Curve


Girder A Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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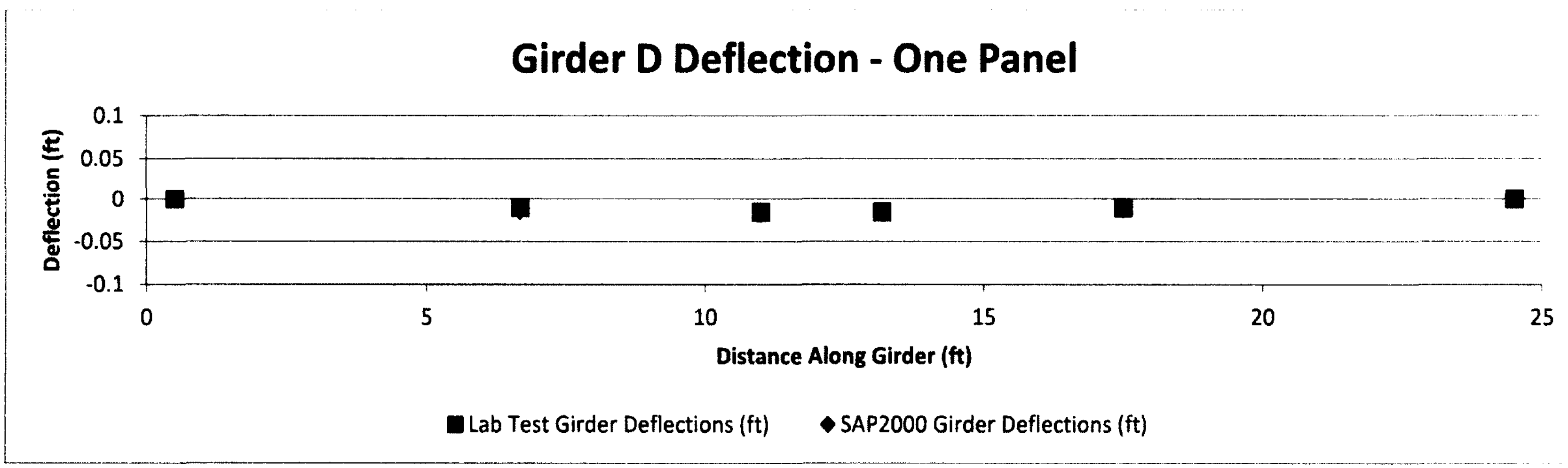
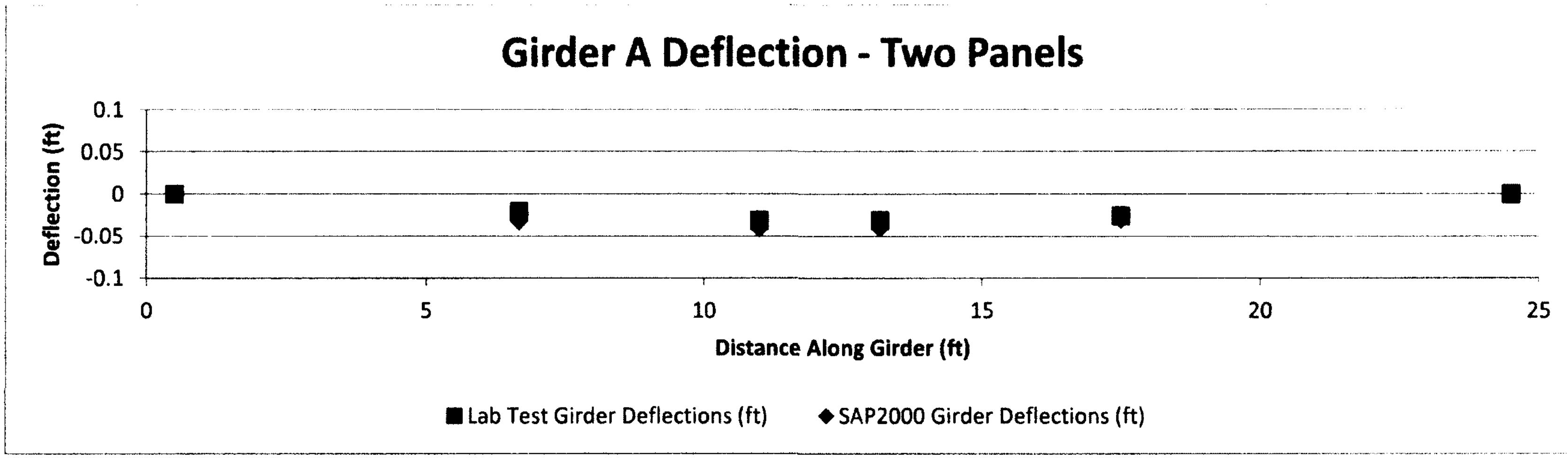
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Deflection - One Panel
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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Deflection (ft)
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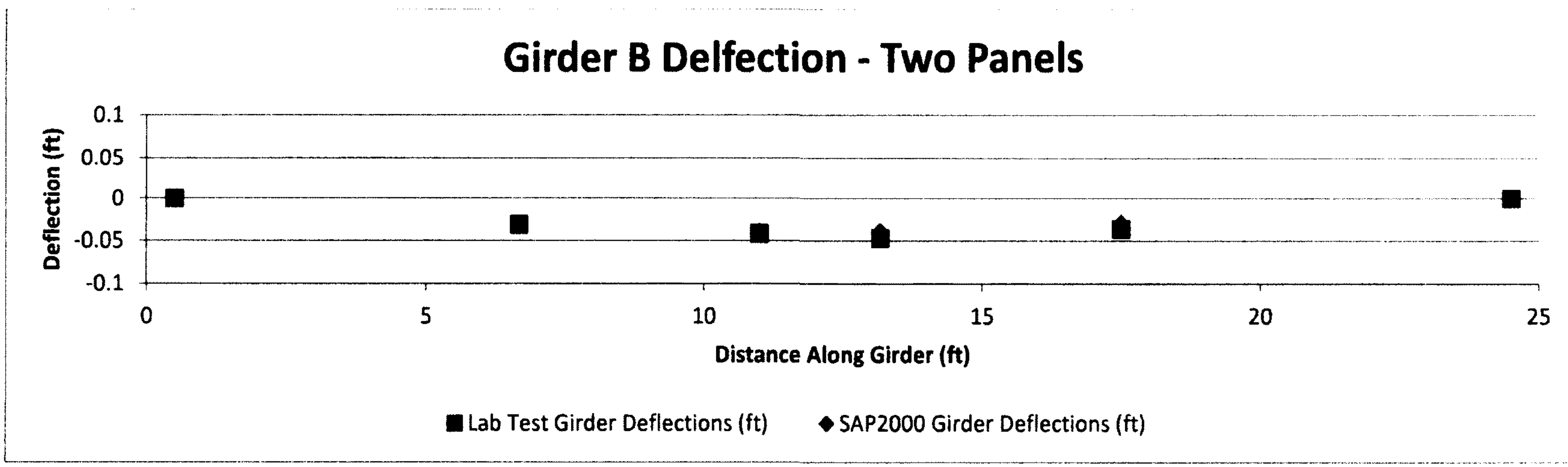
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Delfection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
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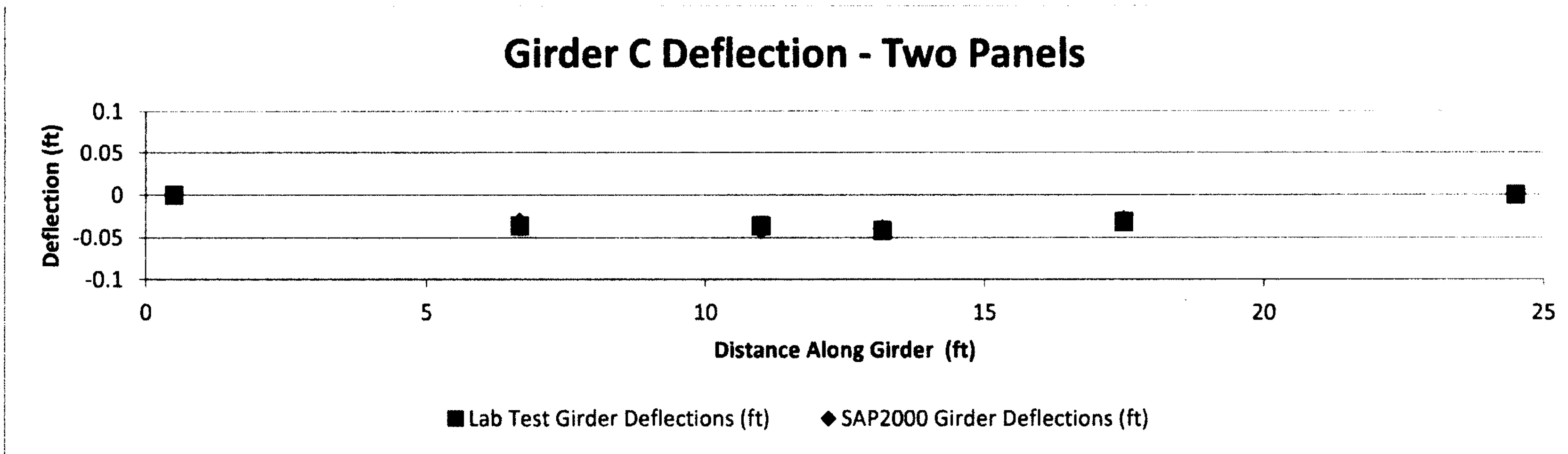
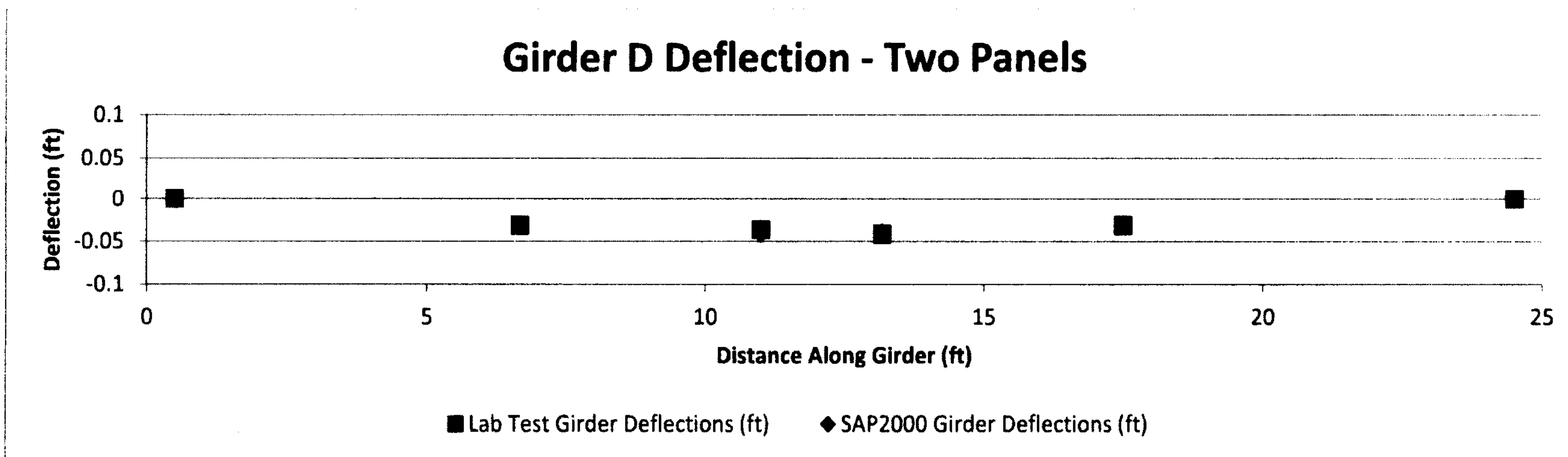
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder C Deflection - Two Panels
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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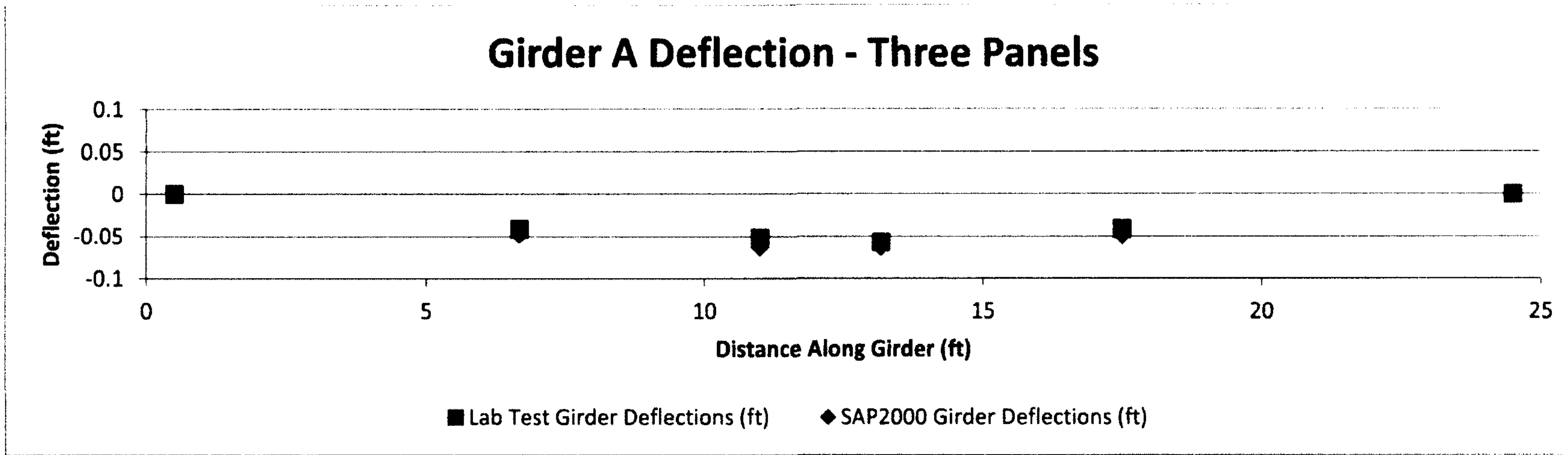
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder A Deflection - Three Panels
Deflection (ft)
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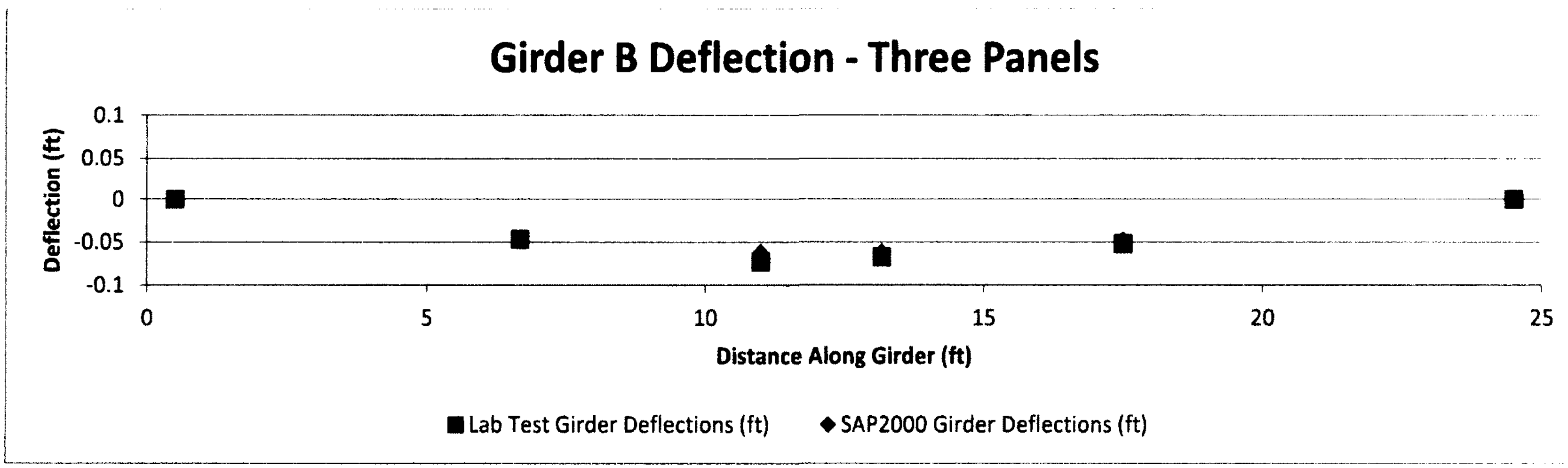
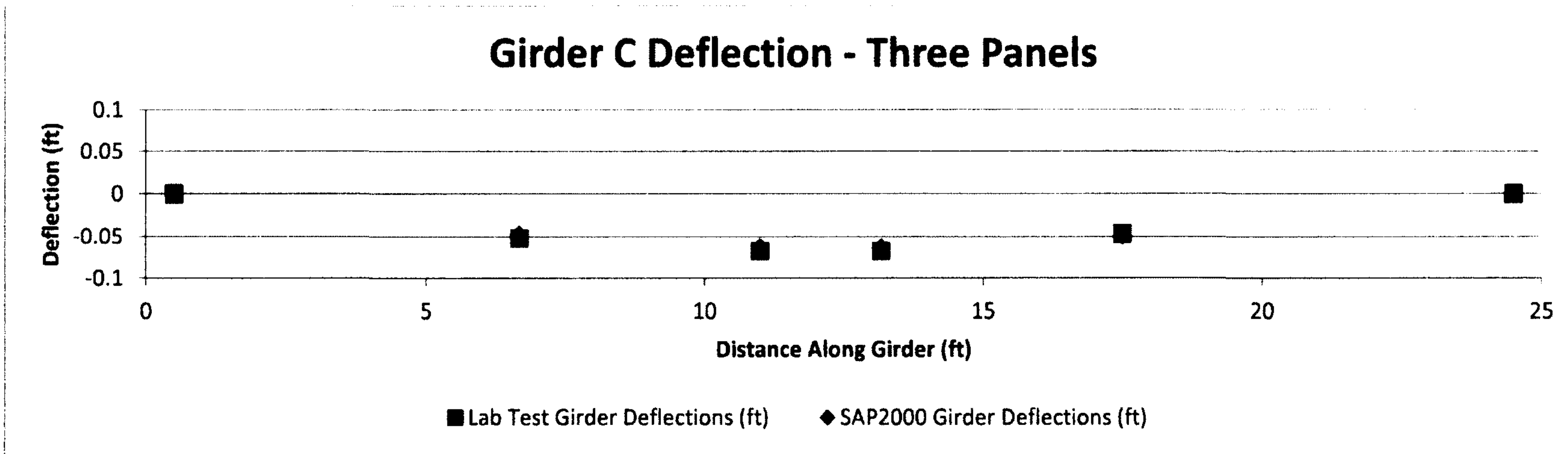
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
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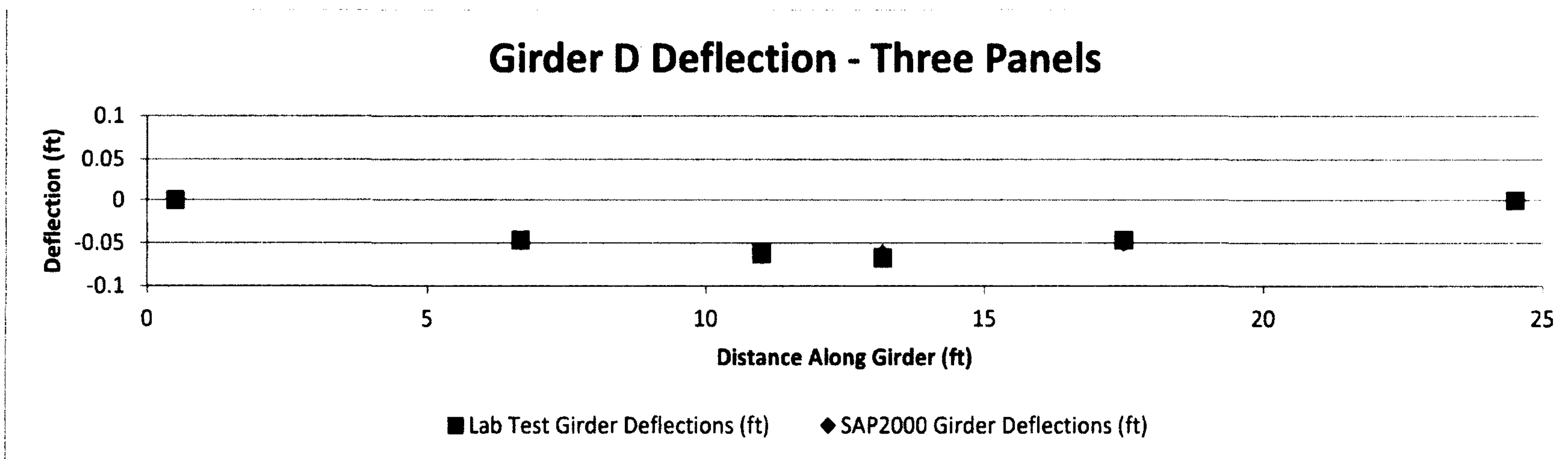
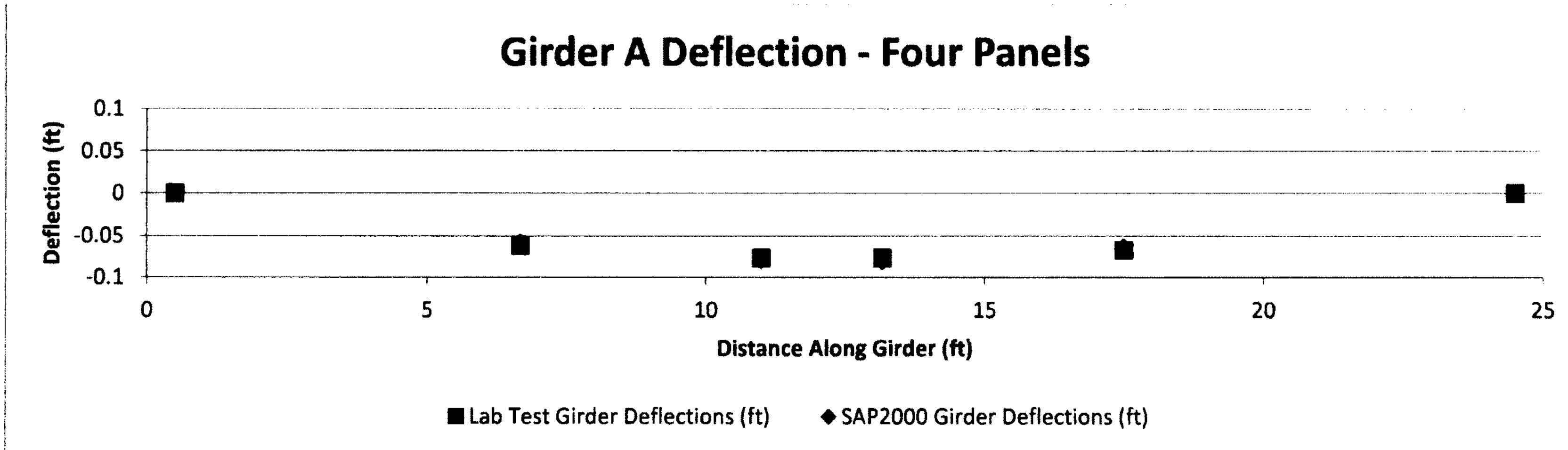
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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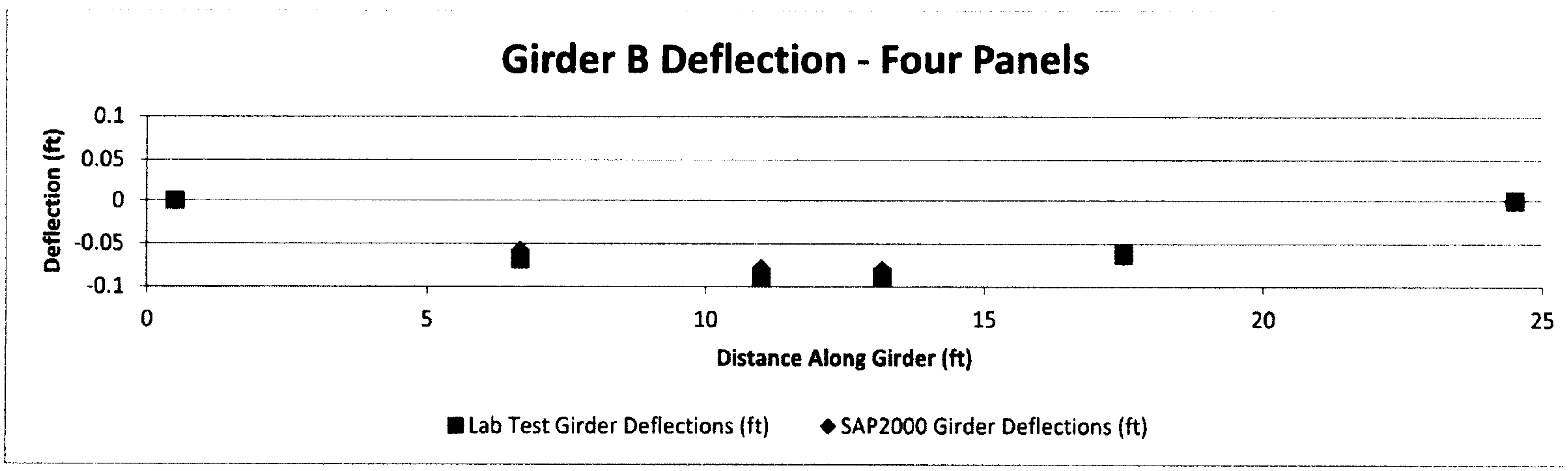
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
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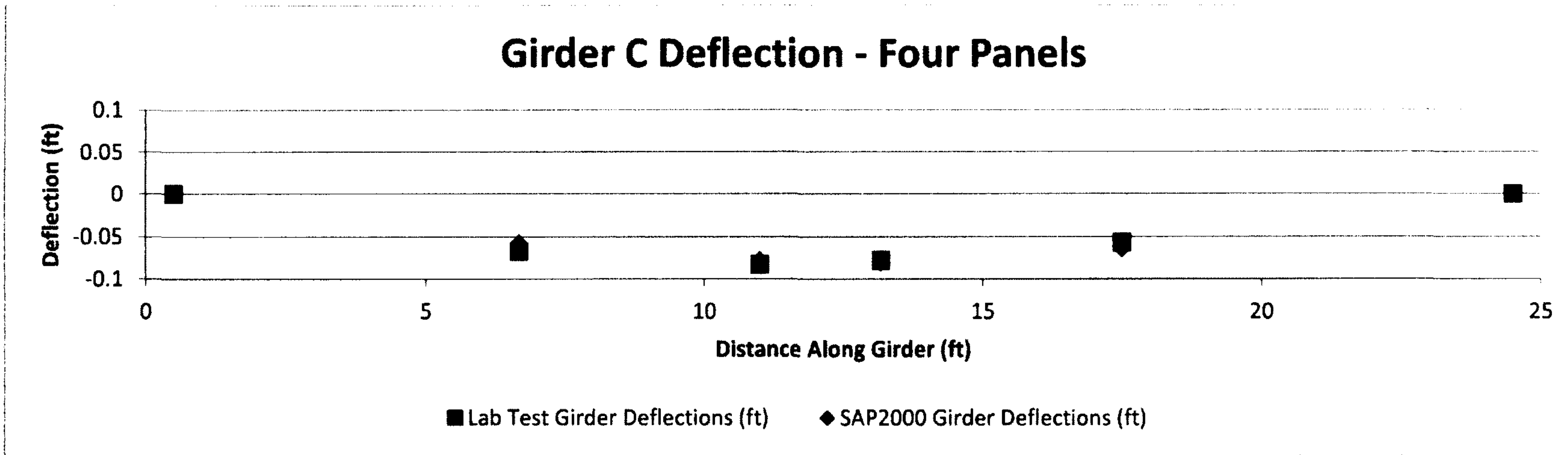
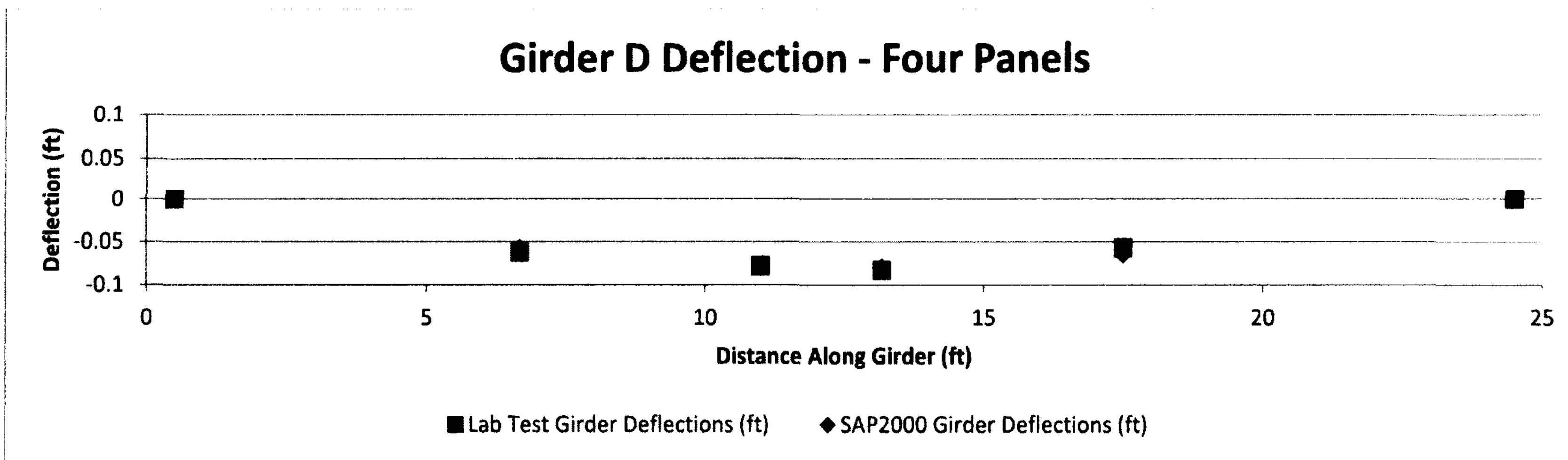
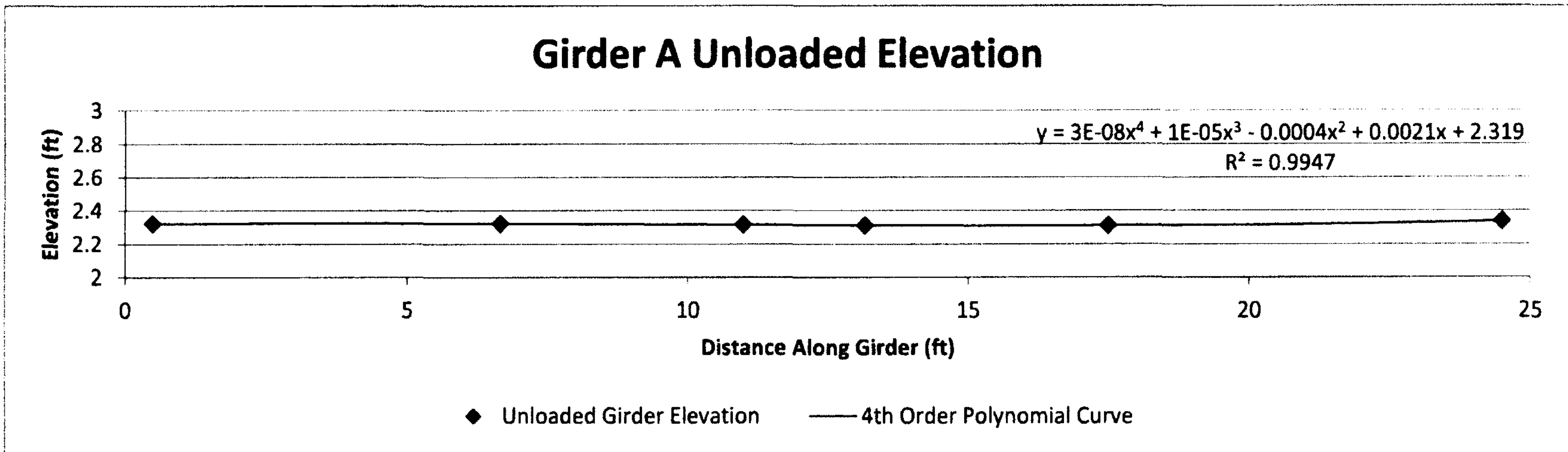
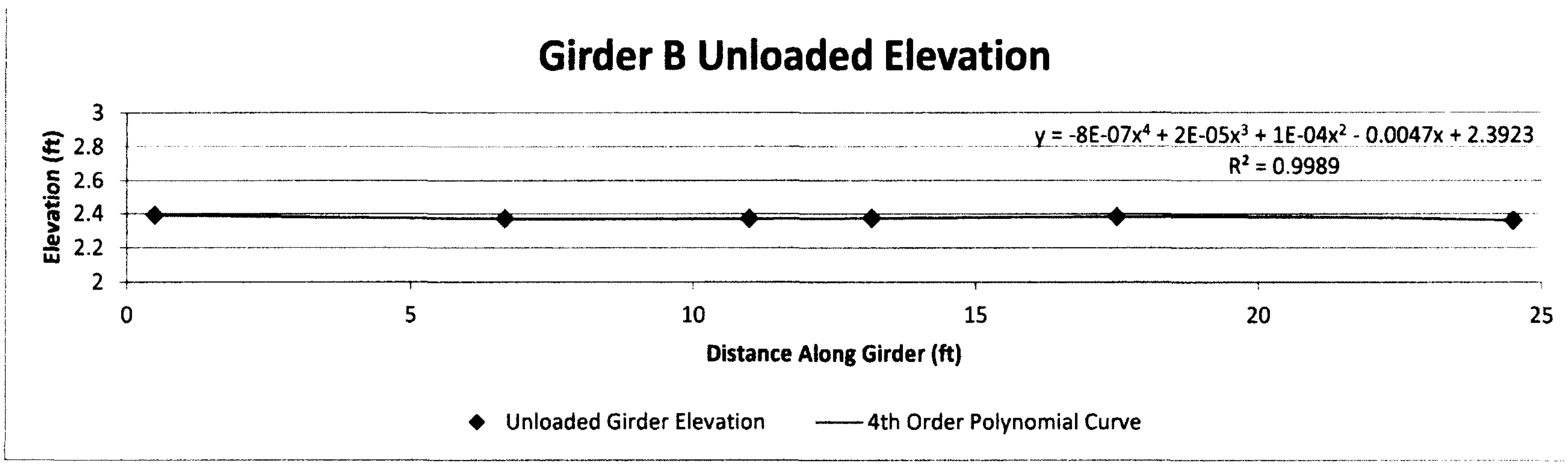
SAP2000 Girder Deflections (ft)

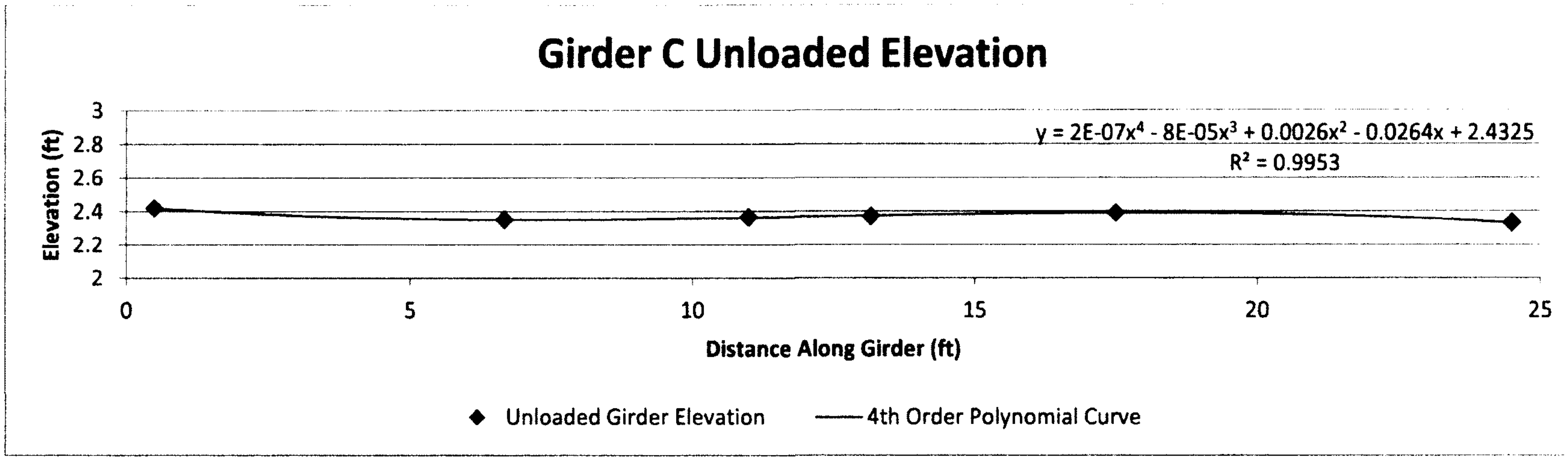

Girder C Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

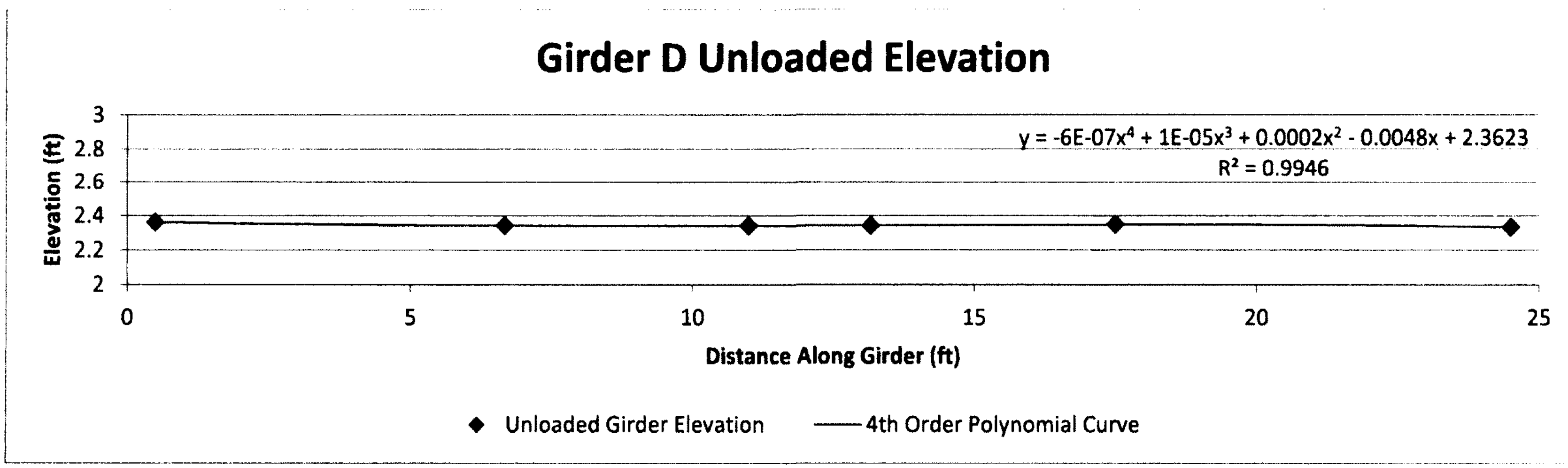

Girder D Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

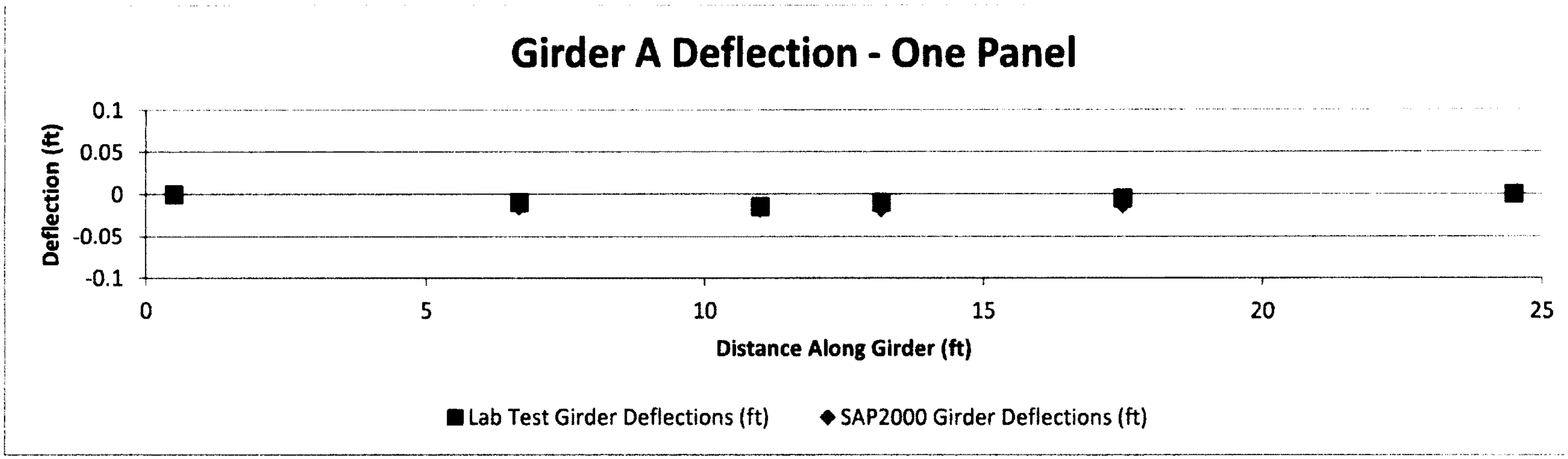
Trial 2 Girder Deflection Graphs

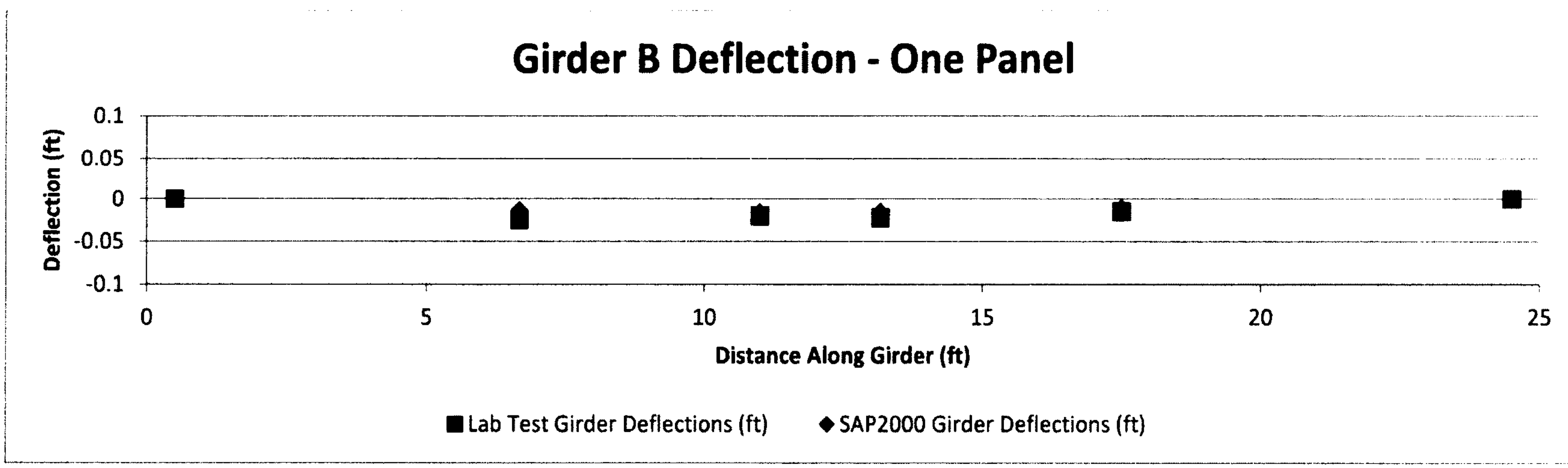


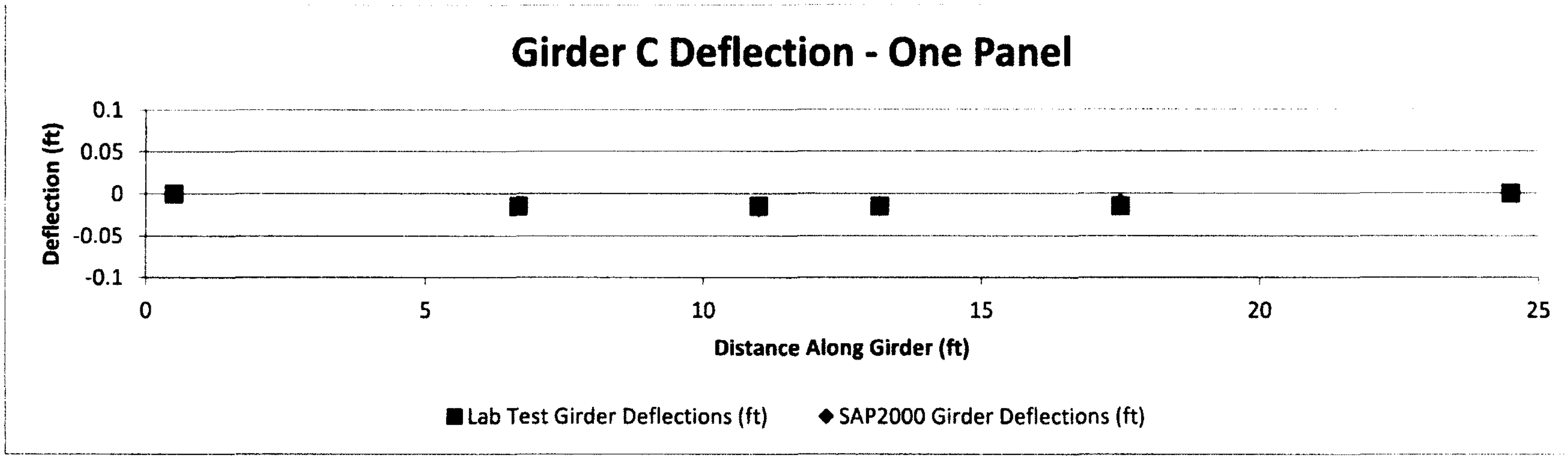


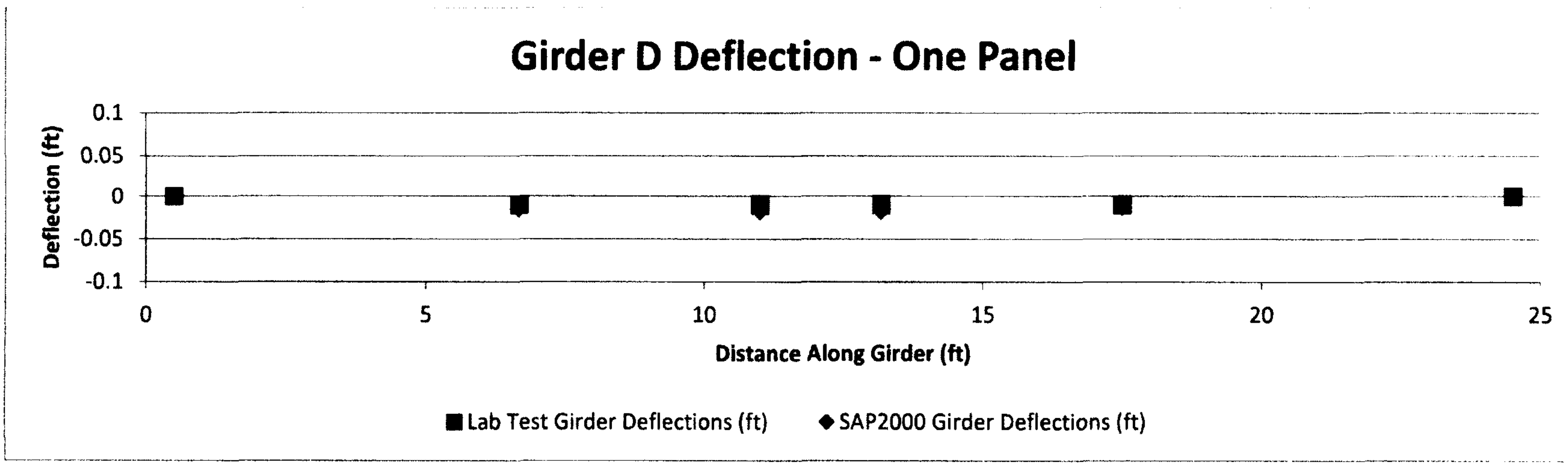

Girder C Unloaded Elevation
y = 2E-07x⁴ - 8E-05x³ + 0.0026x² - 0.0264x + 2.4325
R² = 0.9953
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Unloaded Girder Elevation
4th Order Polynomial Curve

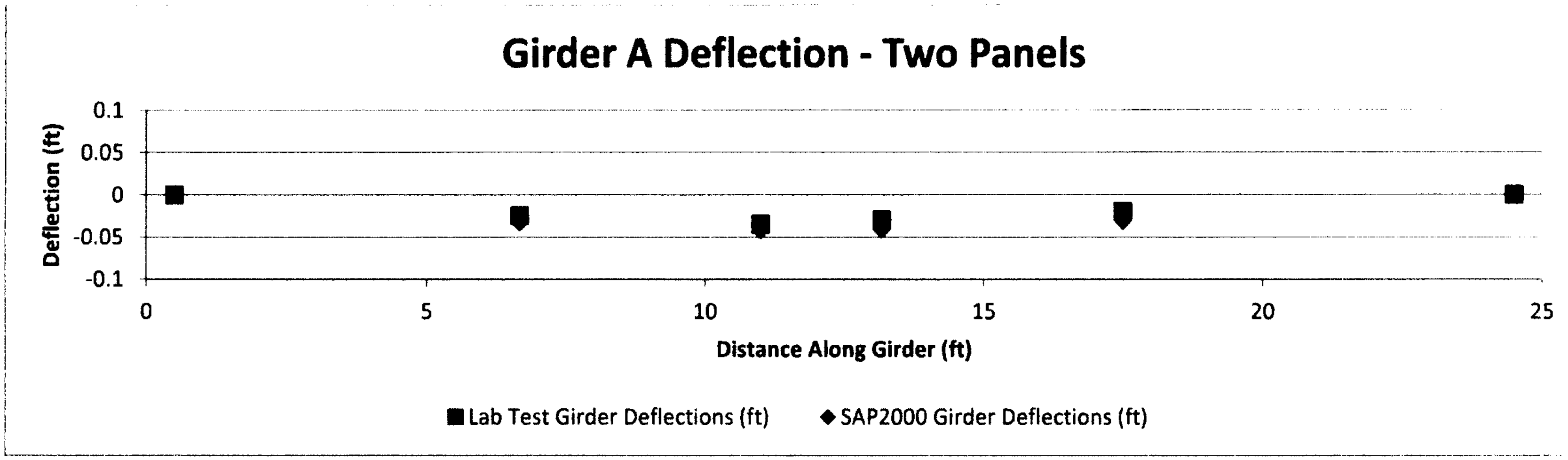

Girder D Unloaded Elevation
y = -6E-07x⁴ + 1E-05x³ + 0.0002x² - 0.0048x + 2.3623
R² = 0.9946
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Unloaded Girder Elevation
4th Order Polynomial Curve

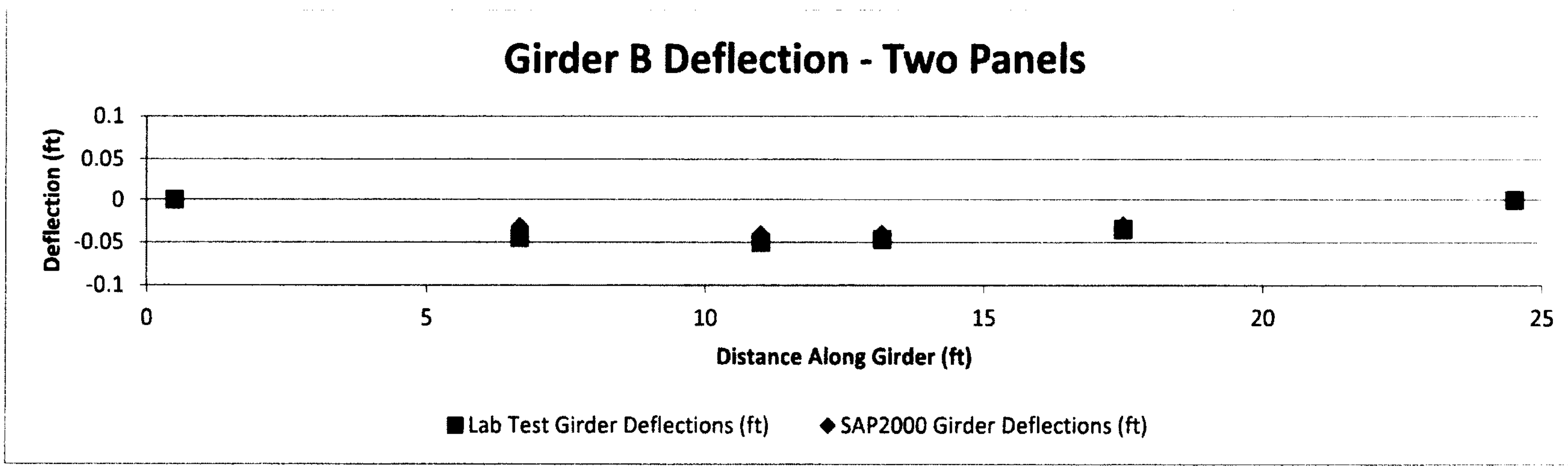

Girder A Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

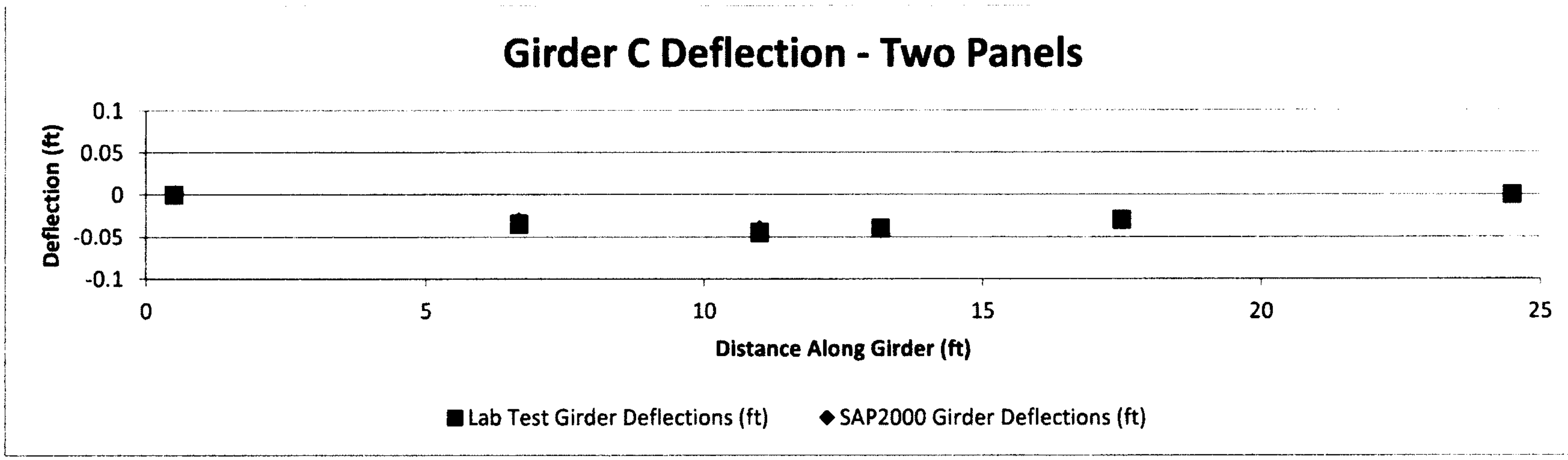

Girder B Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

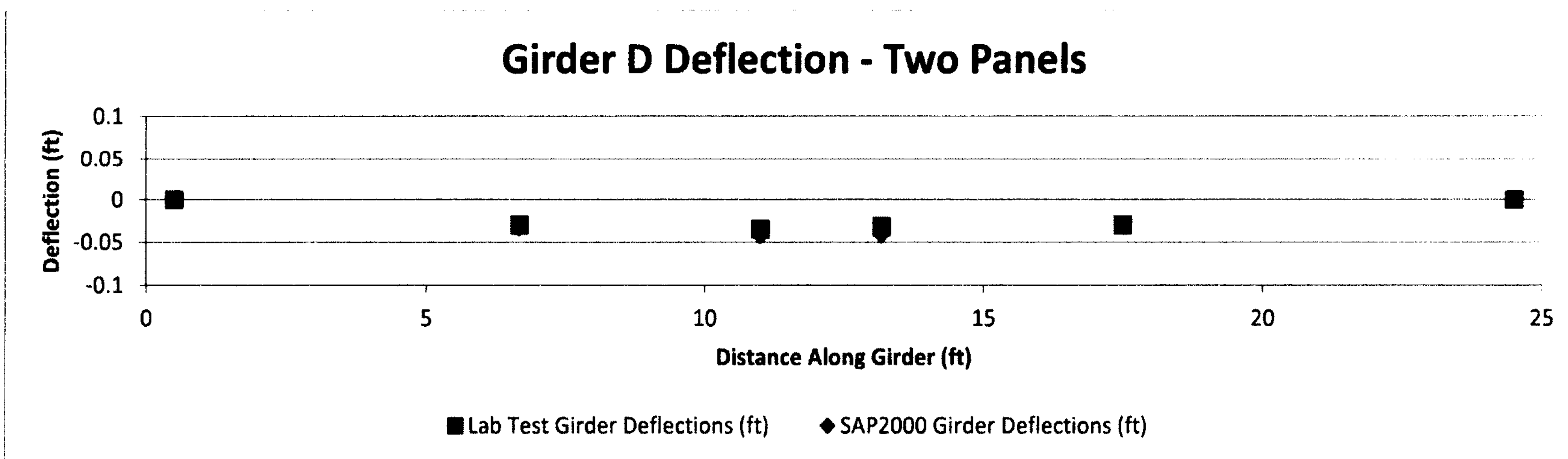

Girder C Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

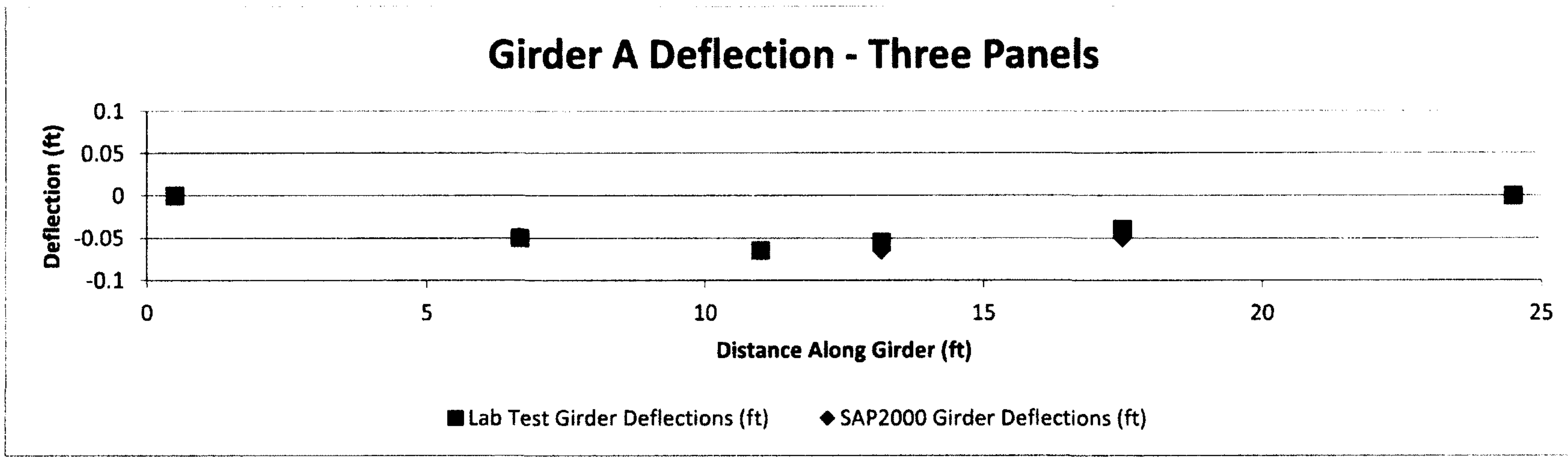

Girder D Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

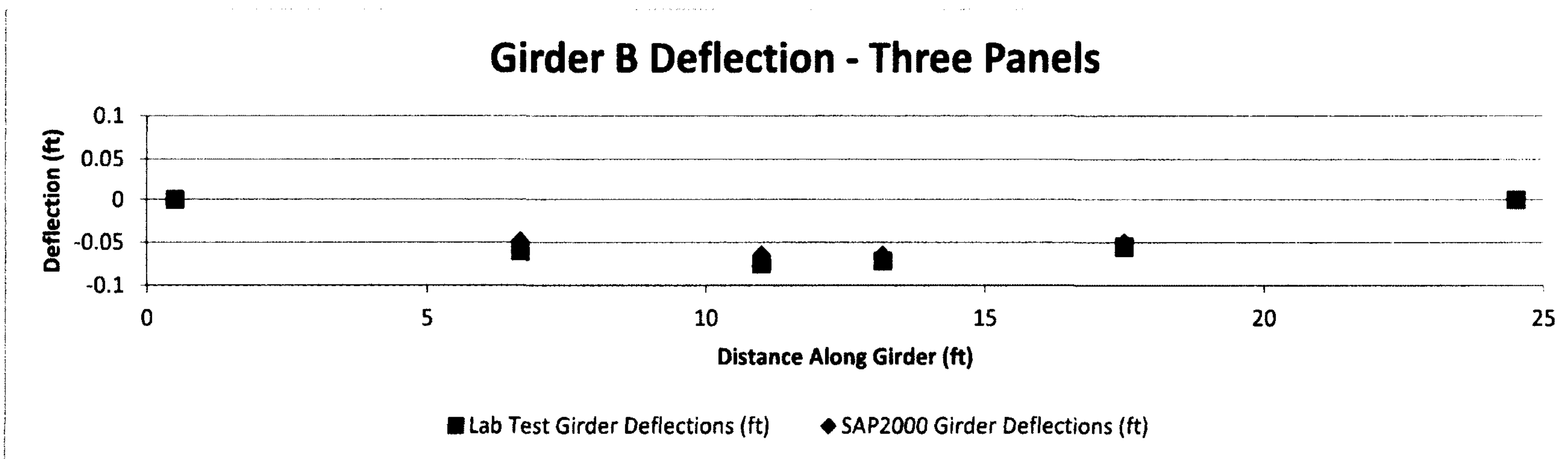

Girder A Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

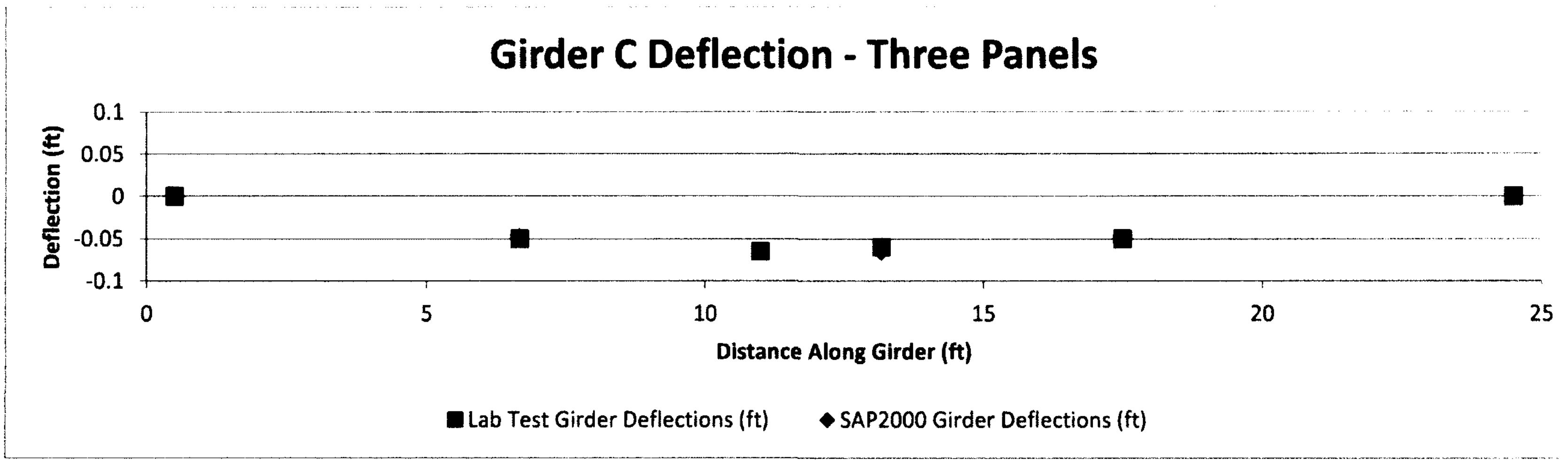

Girder B Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

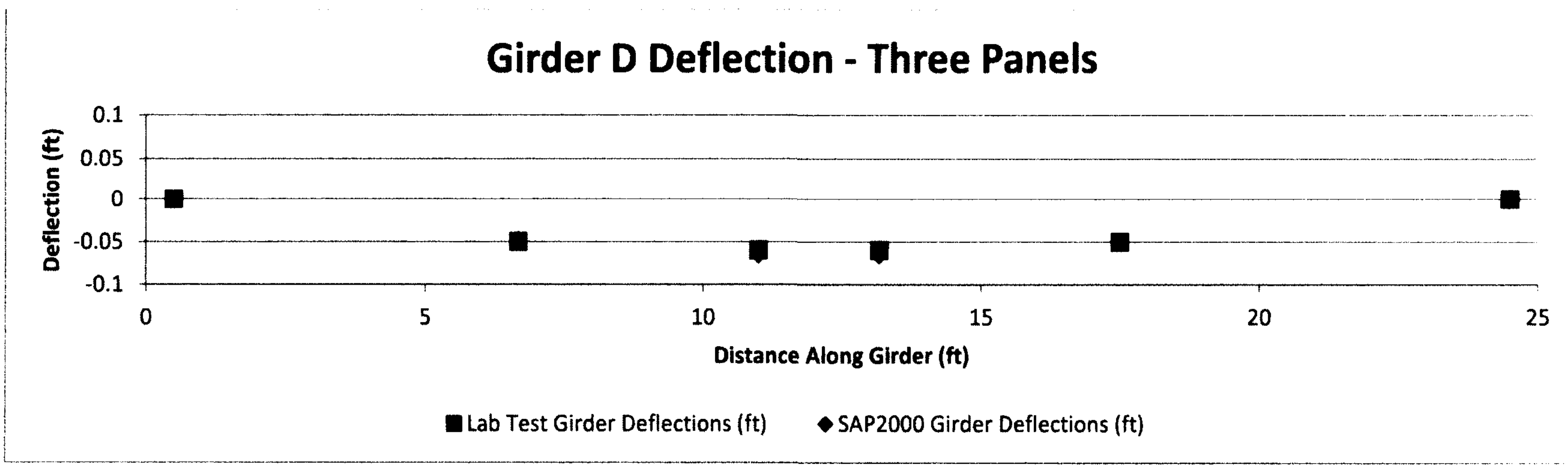

Girder C Deflection - Two Panels
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

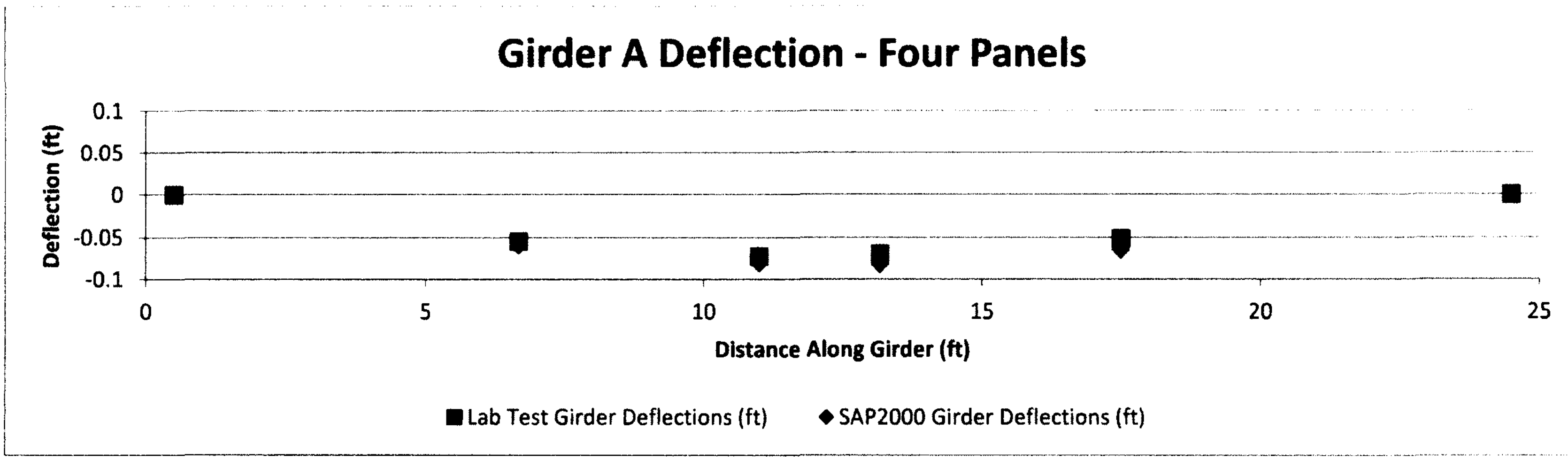

Girder D Deflection - Two Panels
0.1
0.05
0
-0.05
-0.1
Deflection (ft)
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

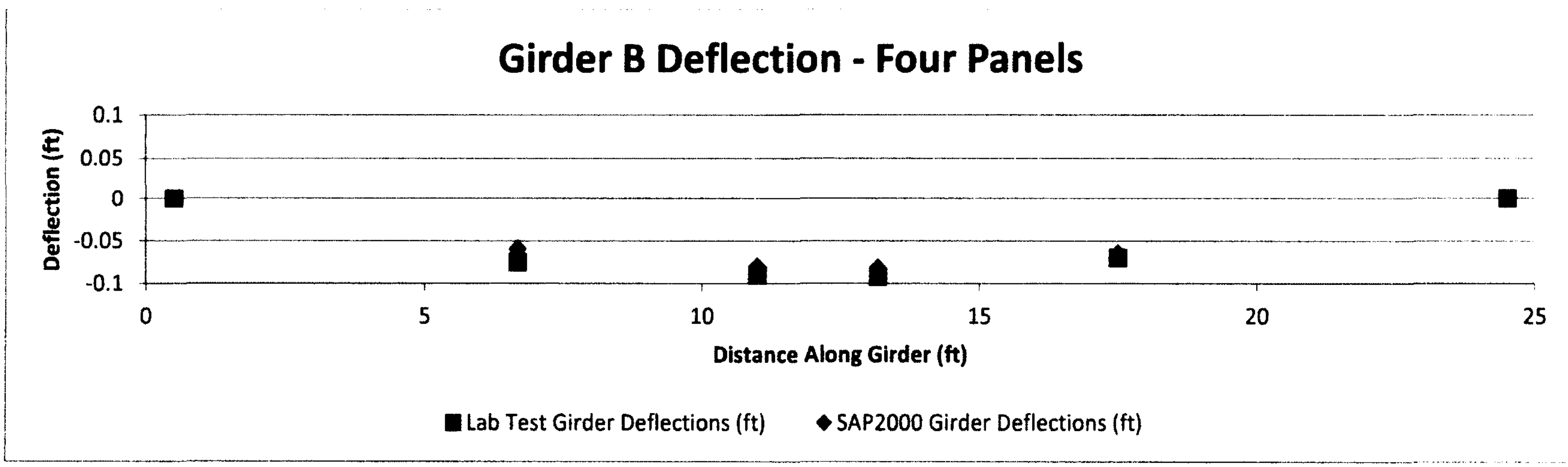

Girder A Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

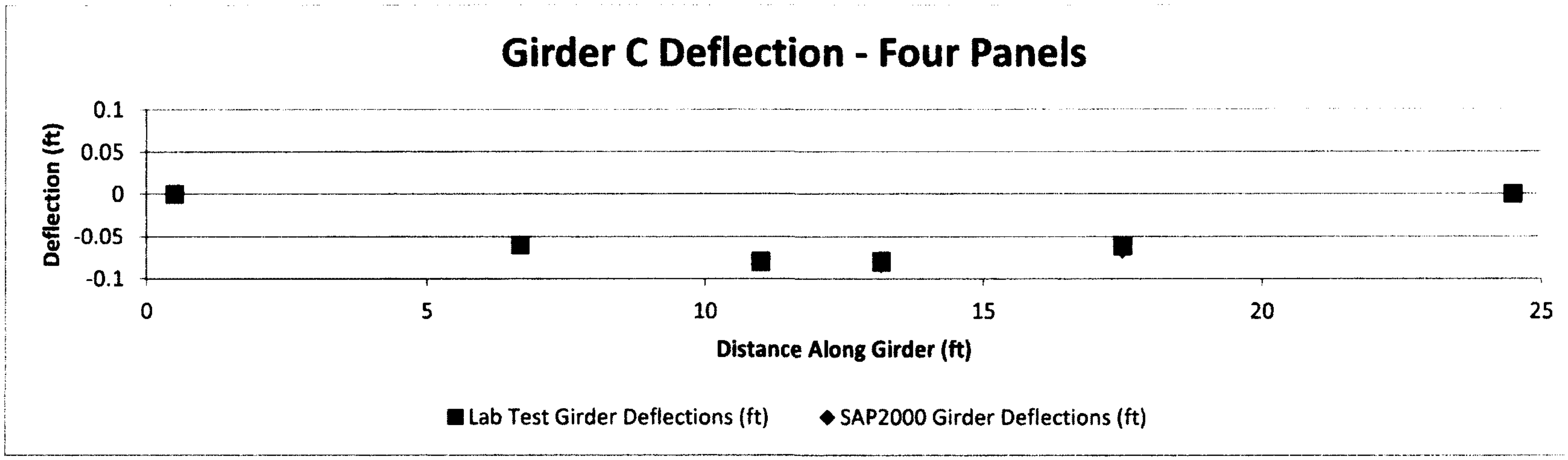

Girder B Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

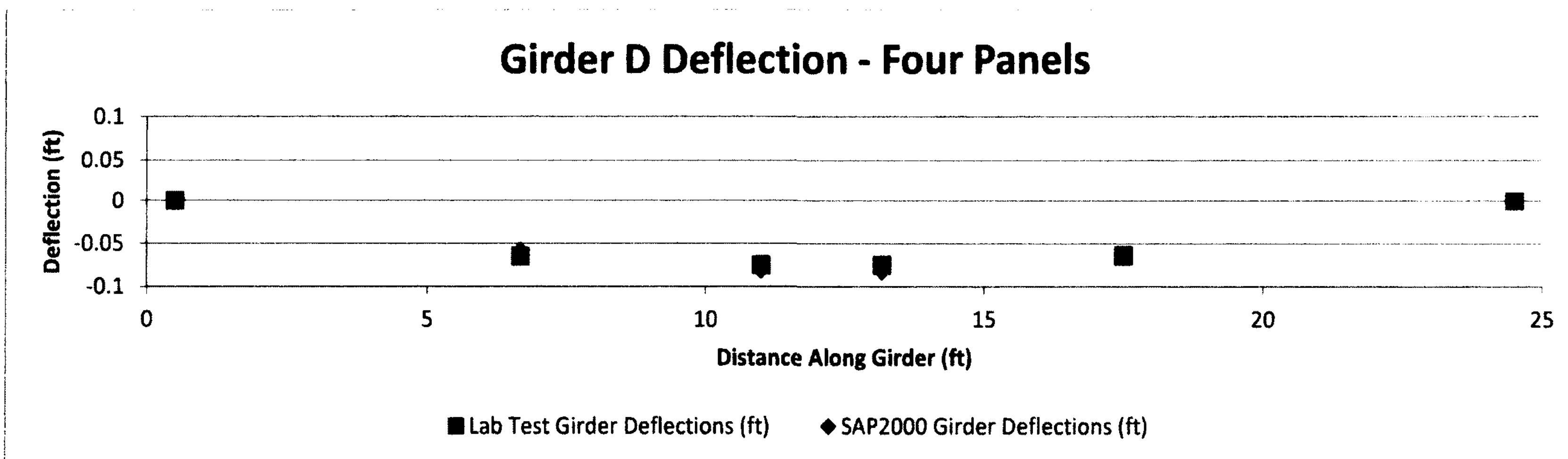

Girder C Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

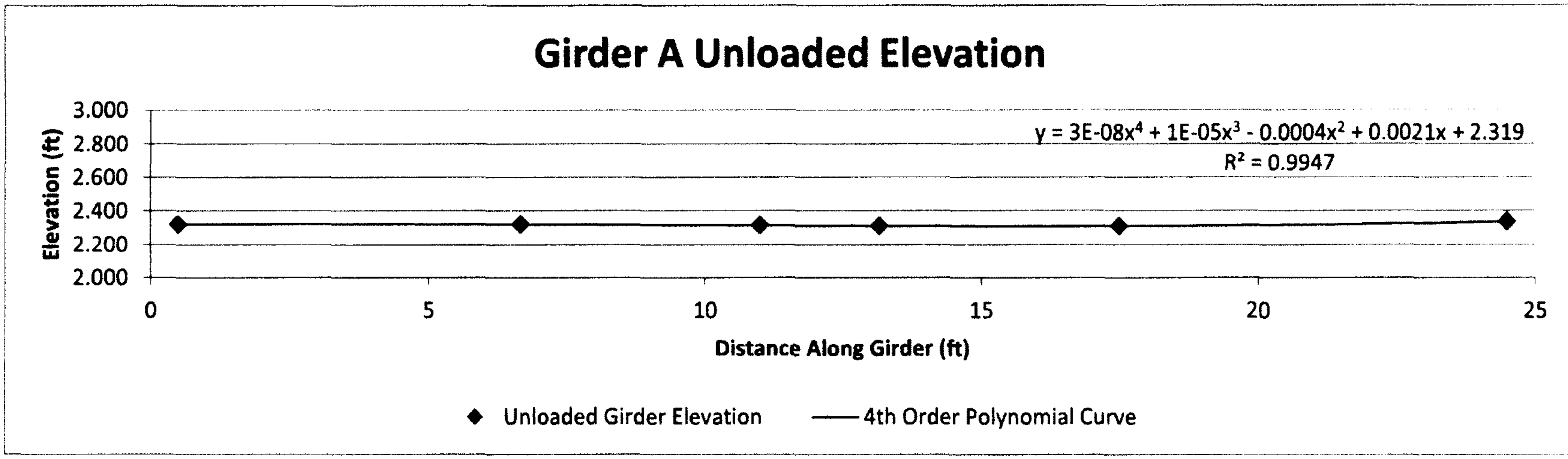

Girder A Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

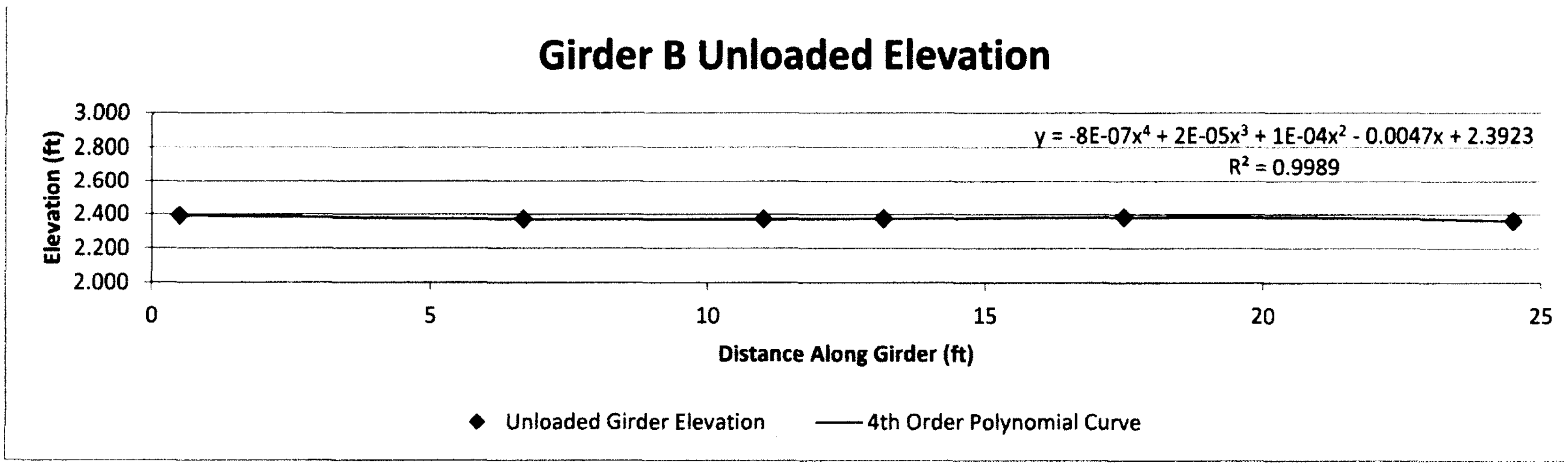

Girder B Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

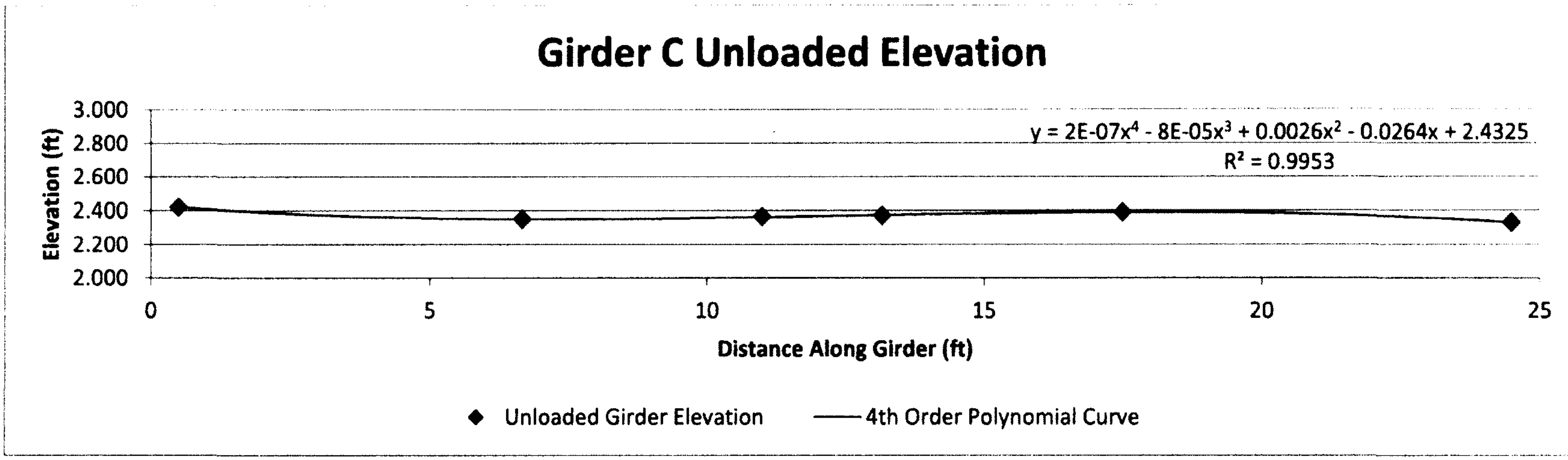

Girder C Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

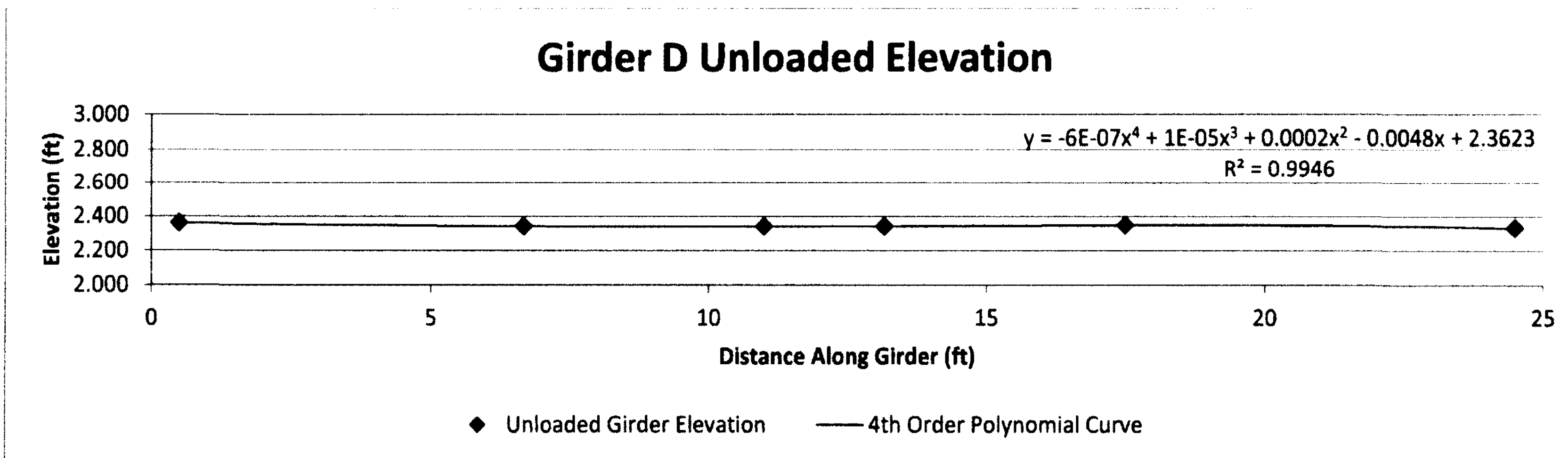

Girder D Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

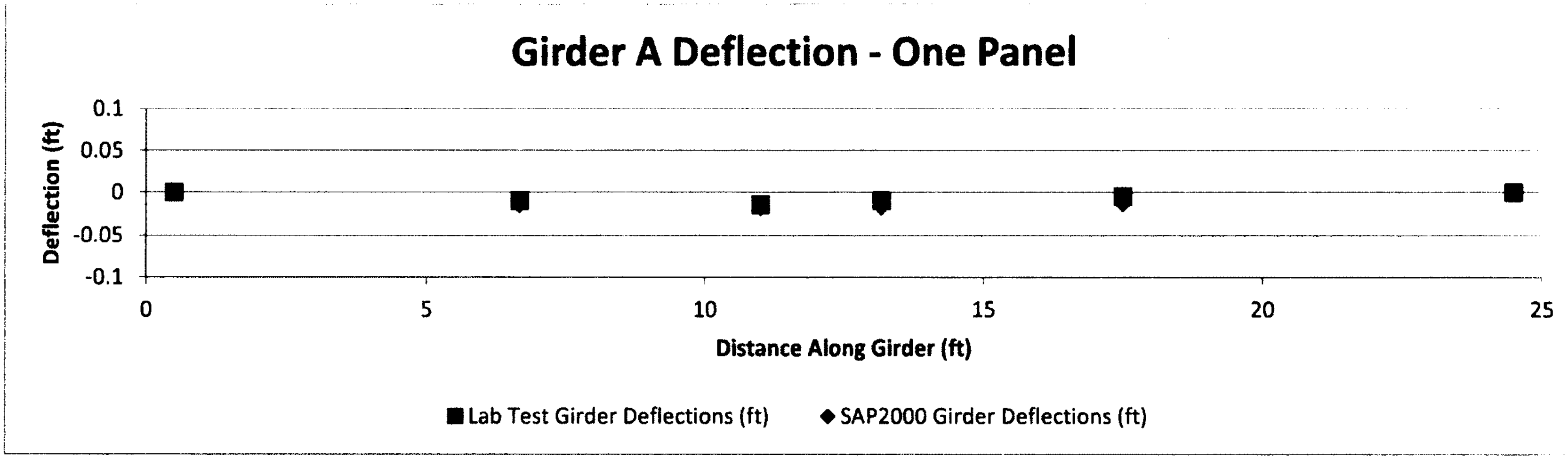
Trial 3 Girder Deflection Graphs

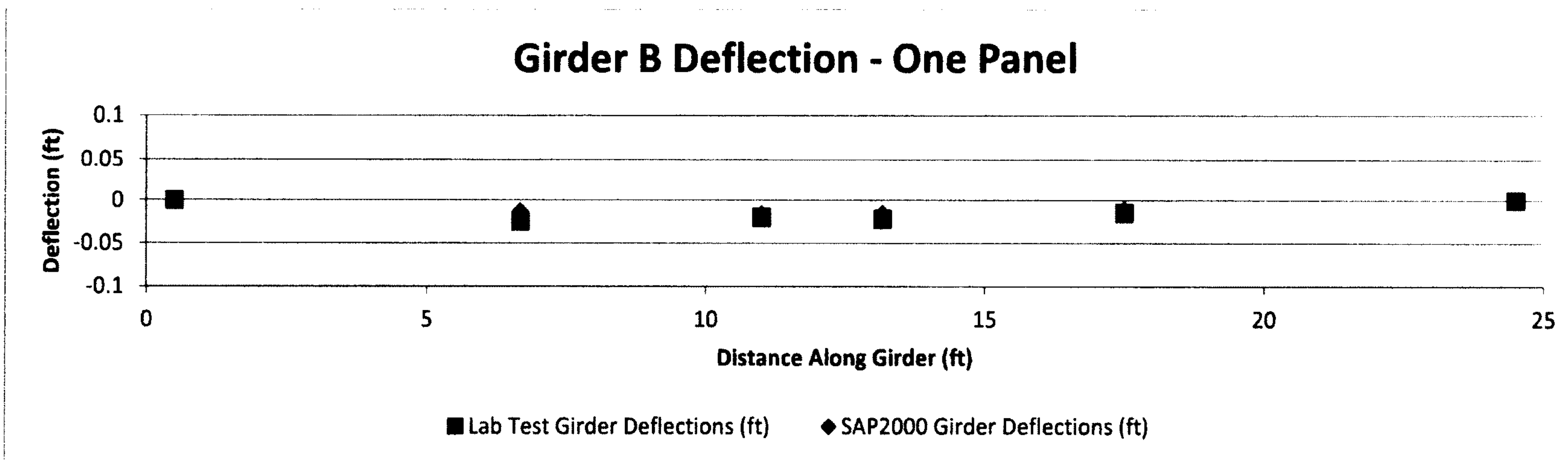


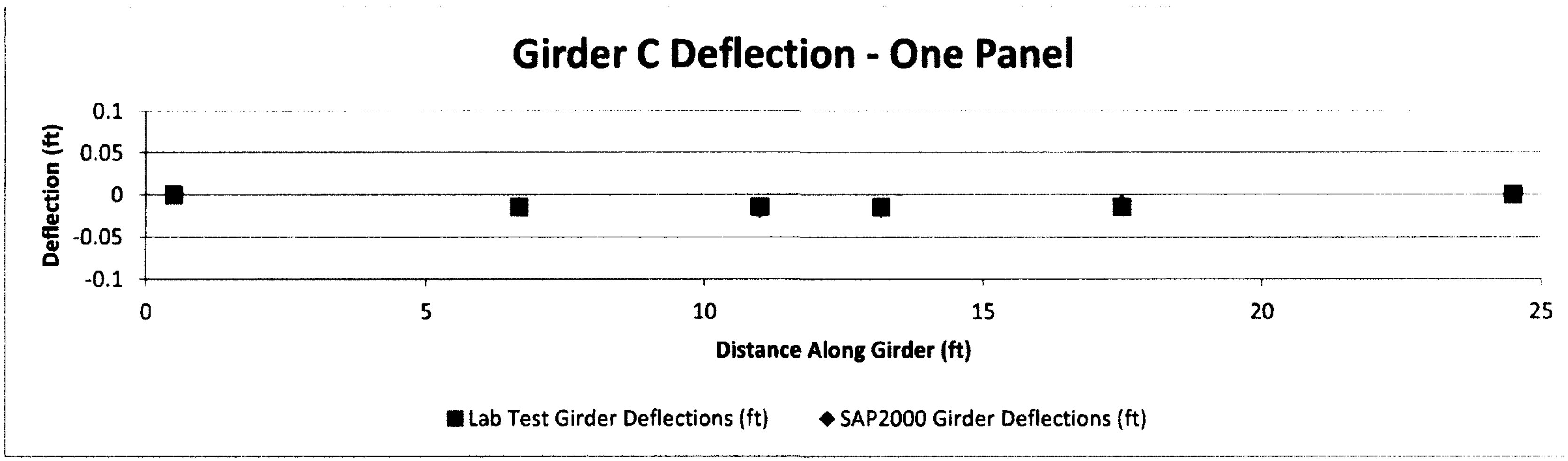


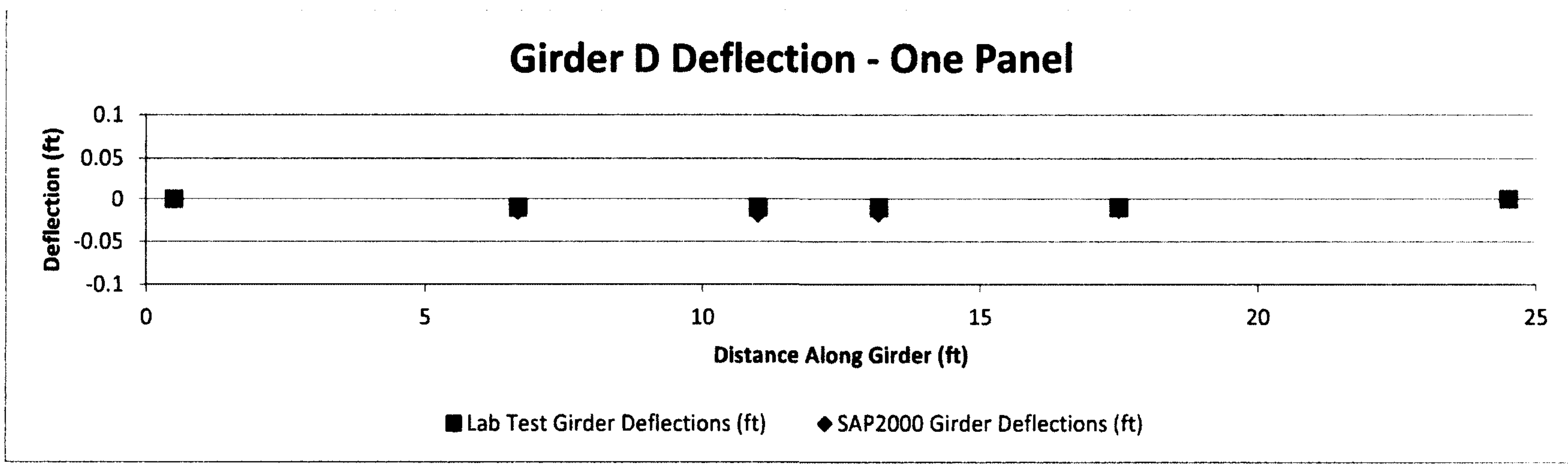

Girder C Unloaded Elevation
Elevation (ft)
3.000
2.800
2.600
2.400
2.200
2.000
0
5
10
15
20
25
Distance Along Girder (ft)
y = 2E-07x⁴ - 8E-05x³ + 0.0026x² - 0.0264x + 2.4325
R² = 0.9953
Unloaded Girder Elevation
4th Order Polynomial Curve

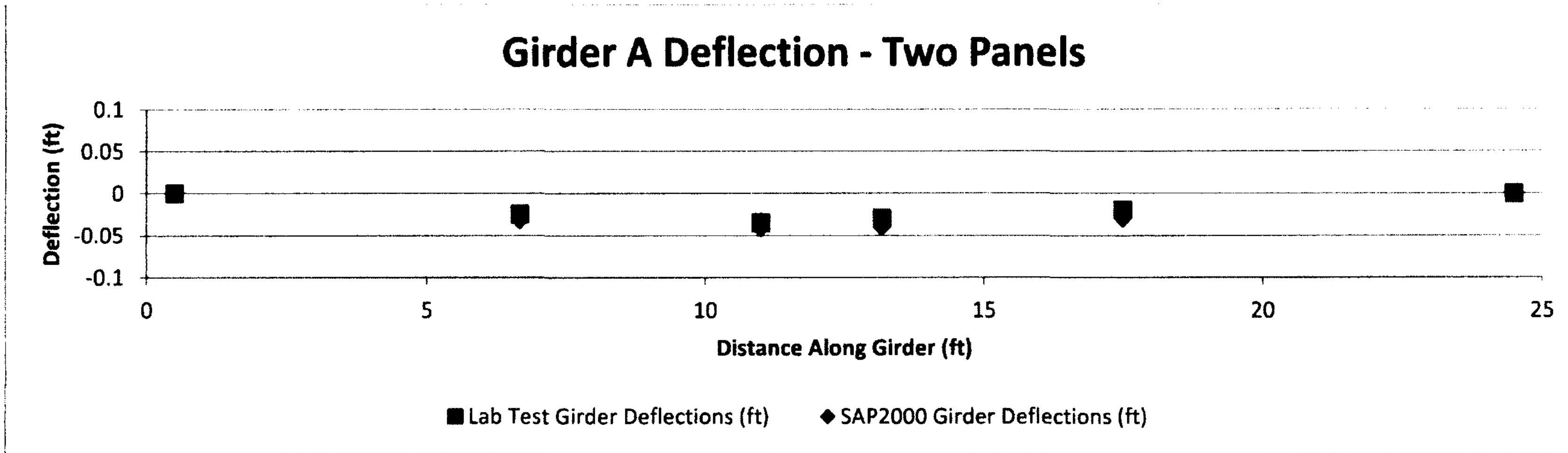

Girder D Unloaded Elevation
Elevation (ft)
3.000
2.800
2.600
2.400
2.200
2.000
0
5
10
15
20
25
Distance Along Girder (ft)
y = -6E-07x⁴ + 1E-05x³ + 0.0002x² - 0.0048x + 2.3623
R² = 0.9946
Unloaded Girder Elevation
4th Order Polynomial Curve

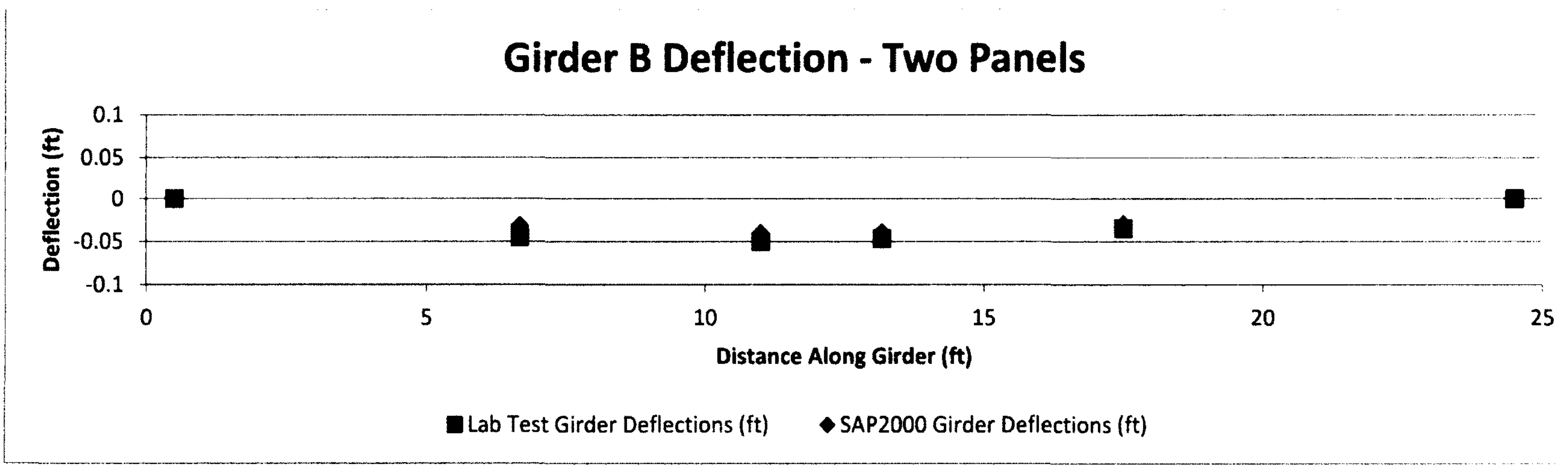

Girder A Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

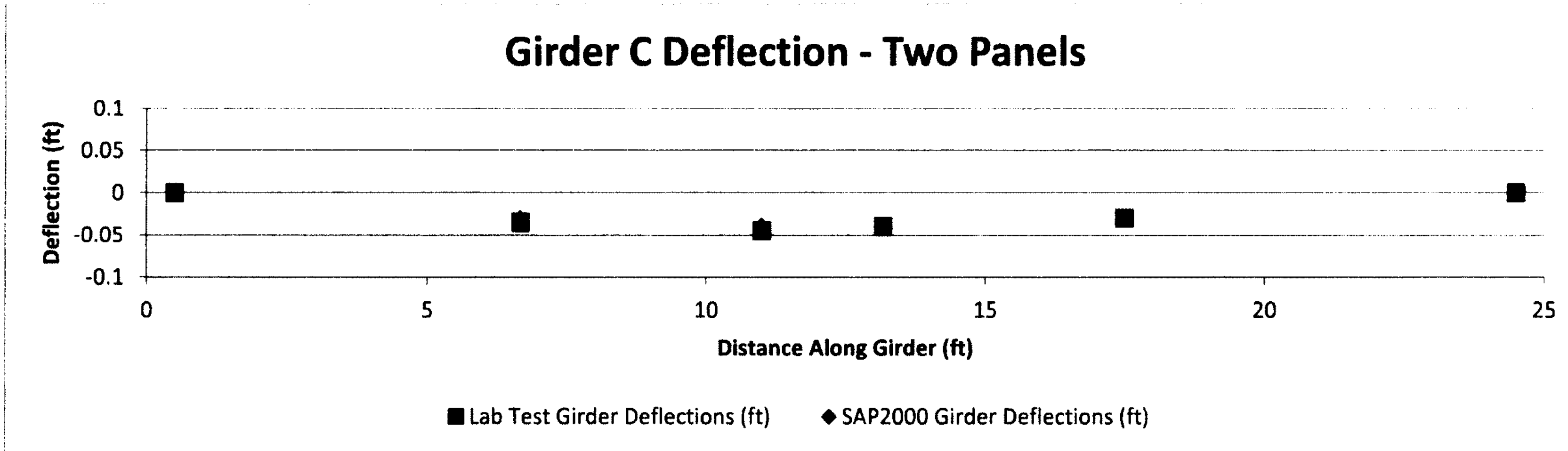

Girder B Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

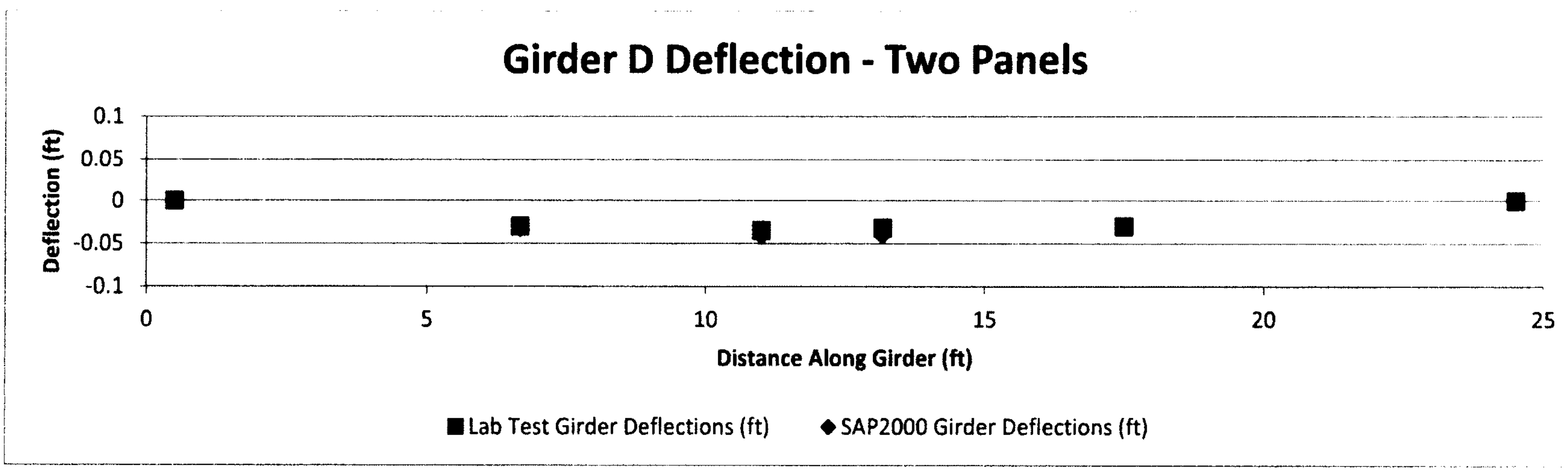

Girder C Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

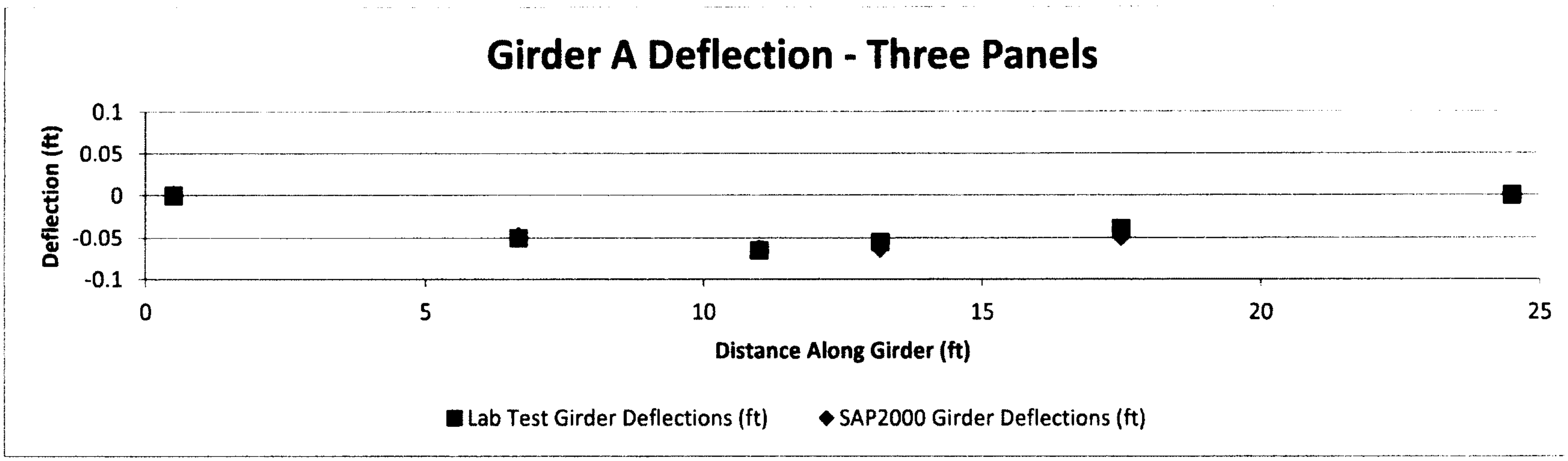

Girder D Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

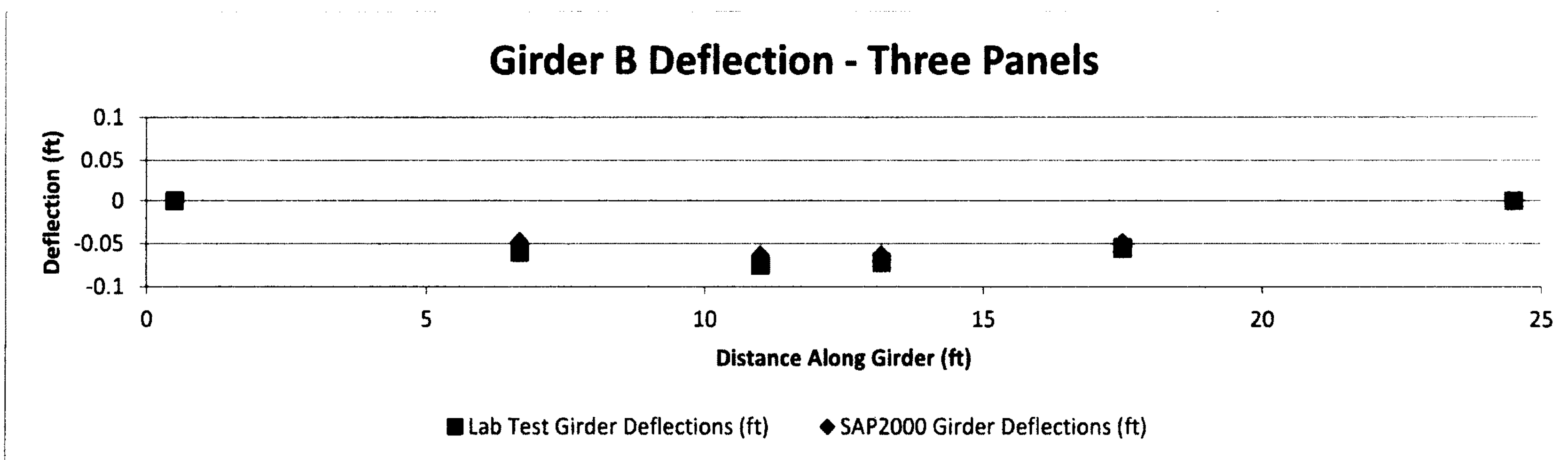

Girder A Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

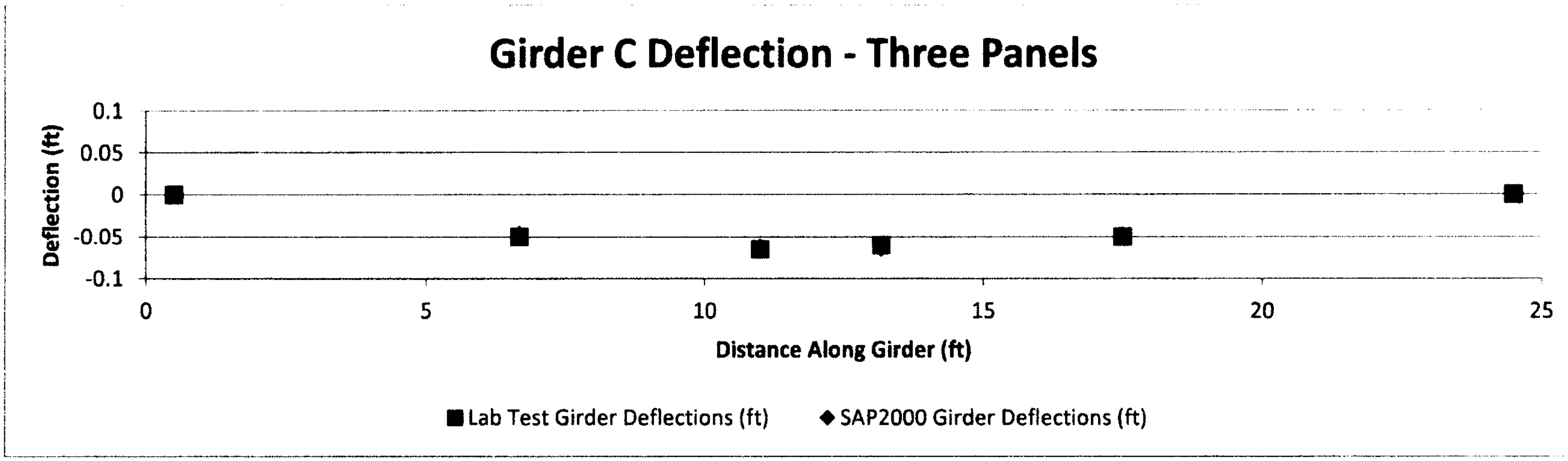

Girder B Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

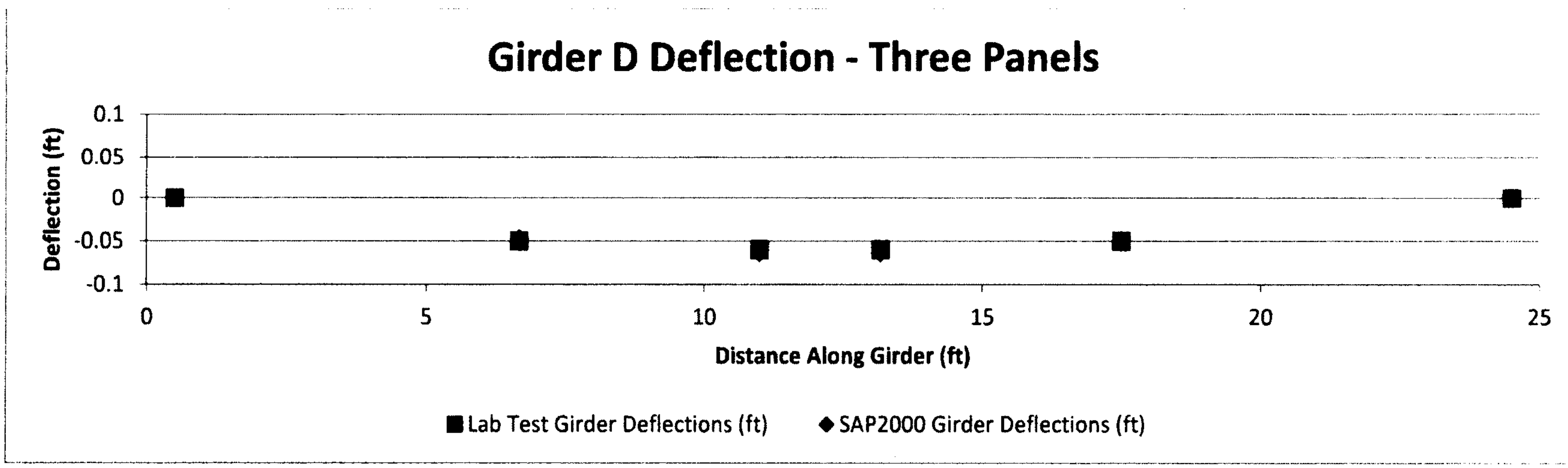

Girder C Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

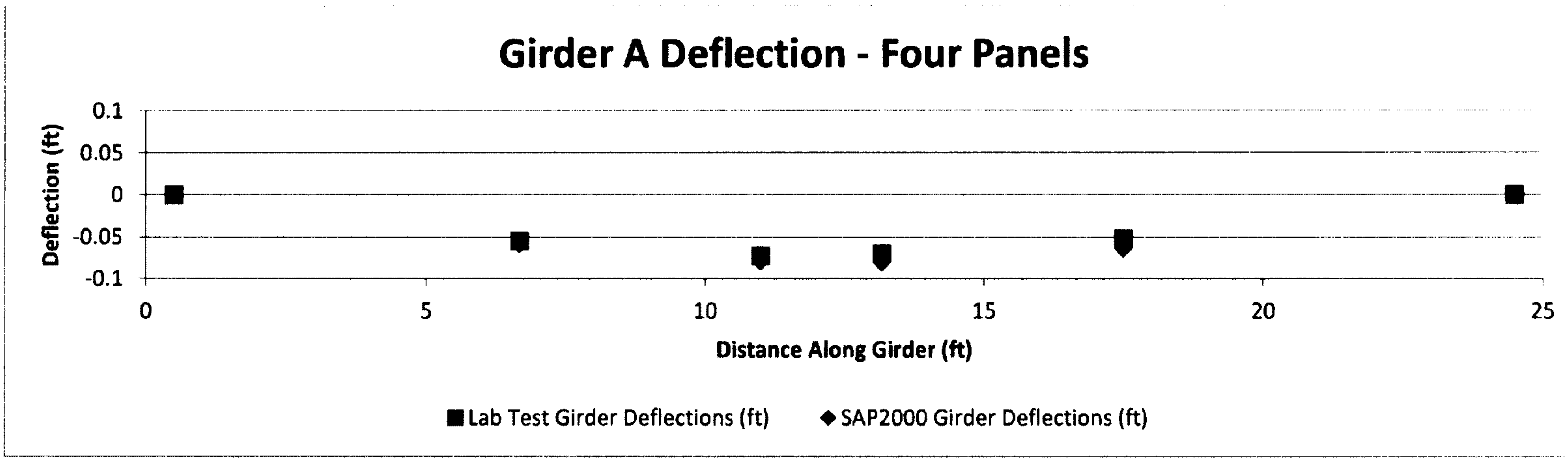

Girder D Deflection - Two Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

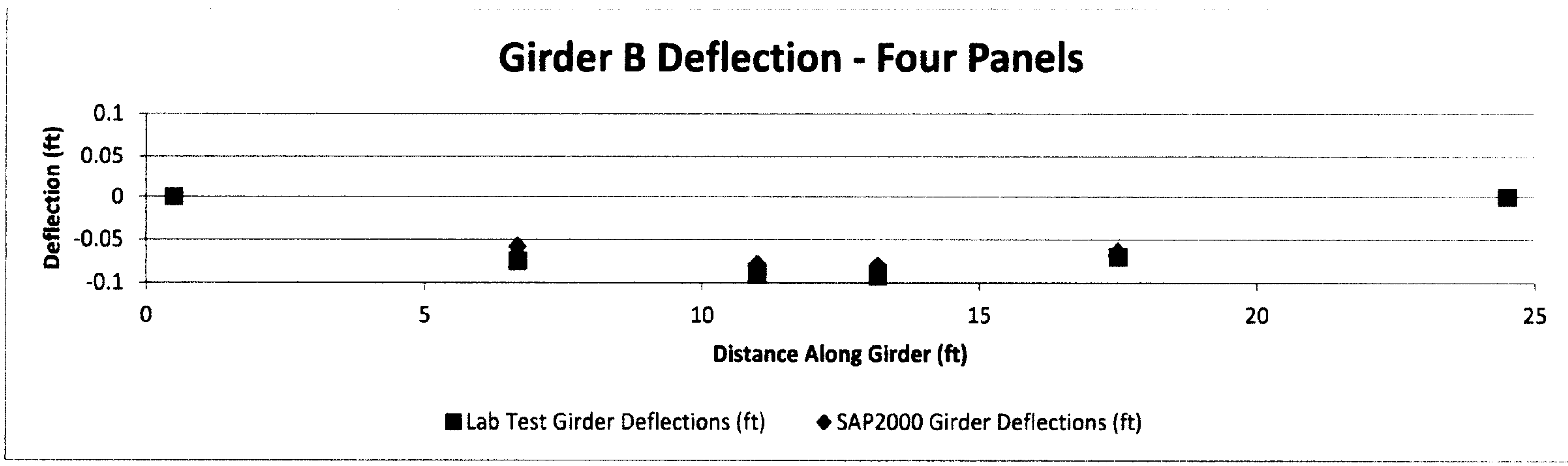

Girder A Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

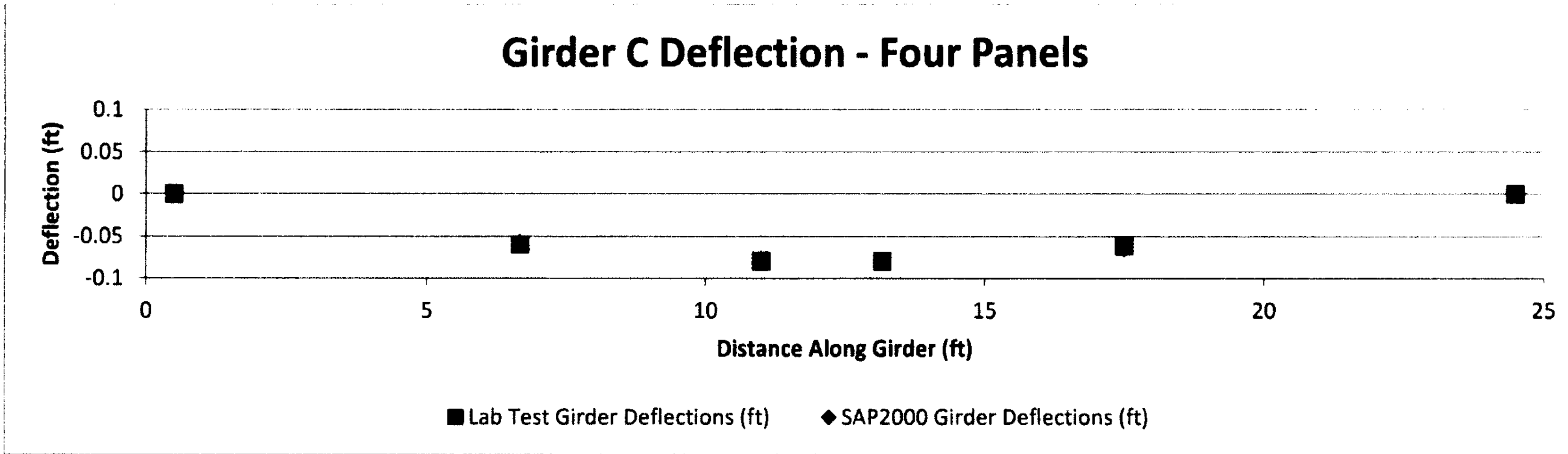

Girder B Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

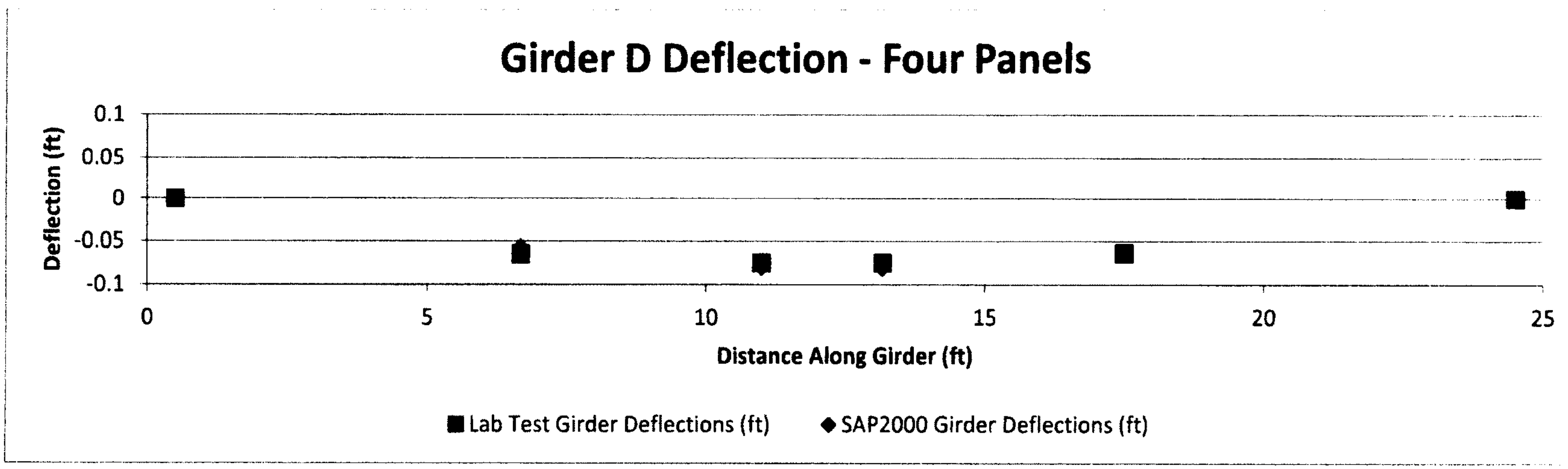

Girder C Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Deflection - Three Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

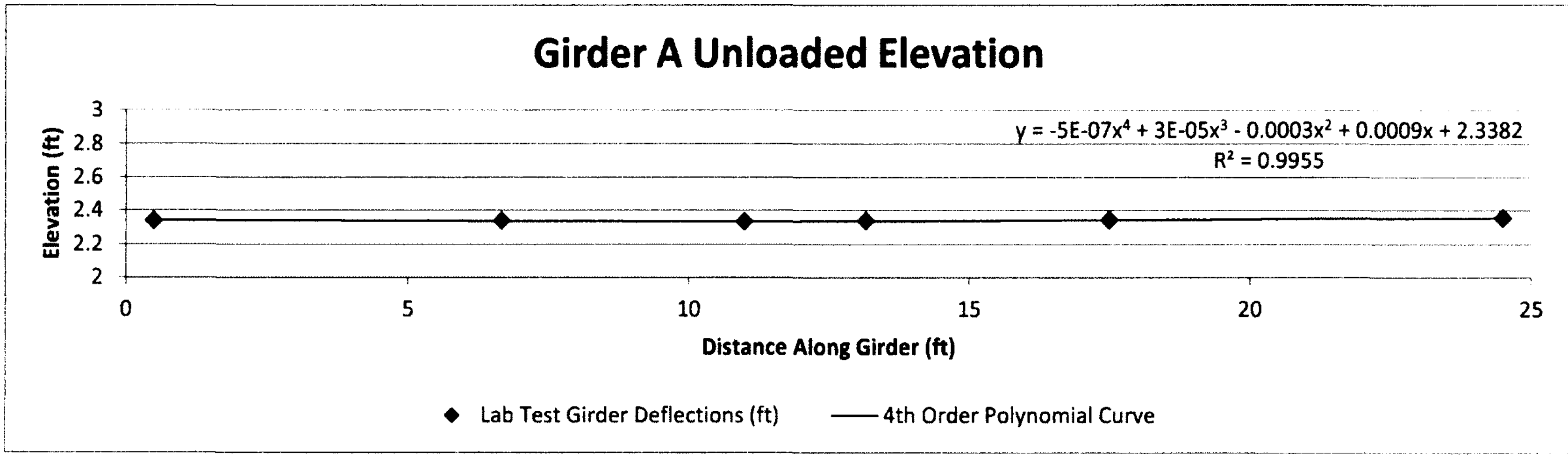

Girder A Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

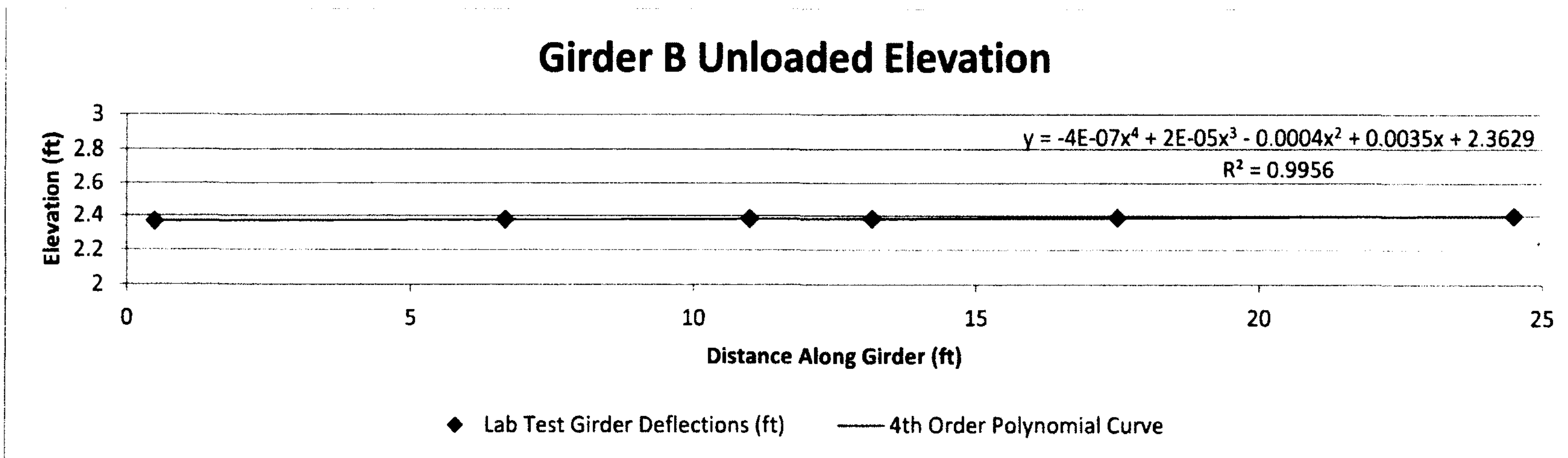

Girder B Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

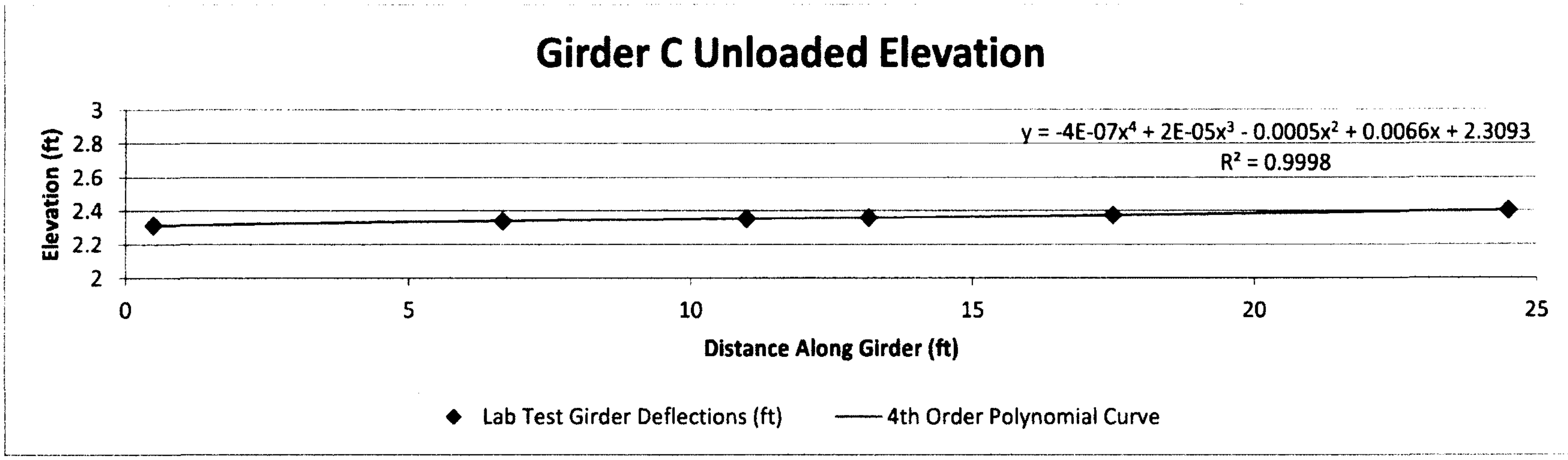

Girder C Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

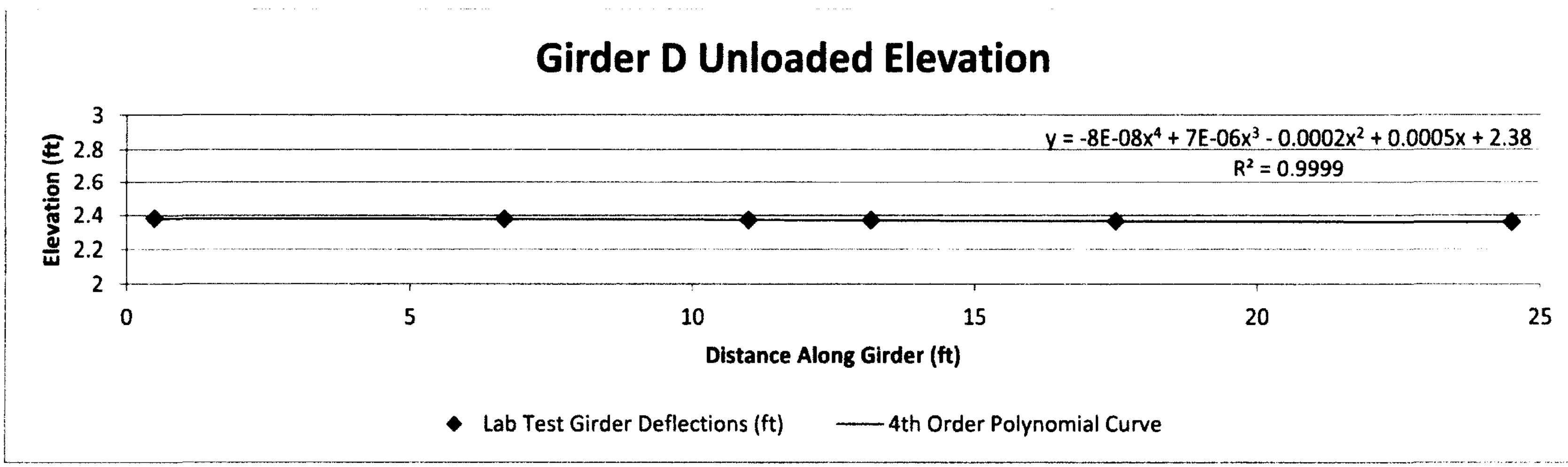

Girder D Deflection - Four Panels
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)

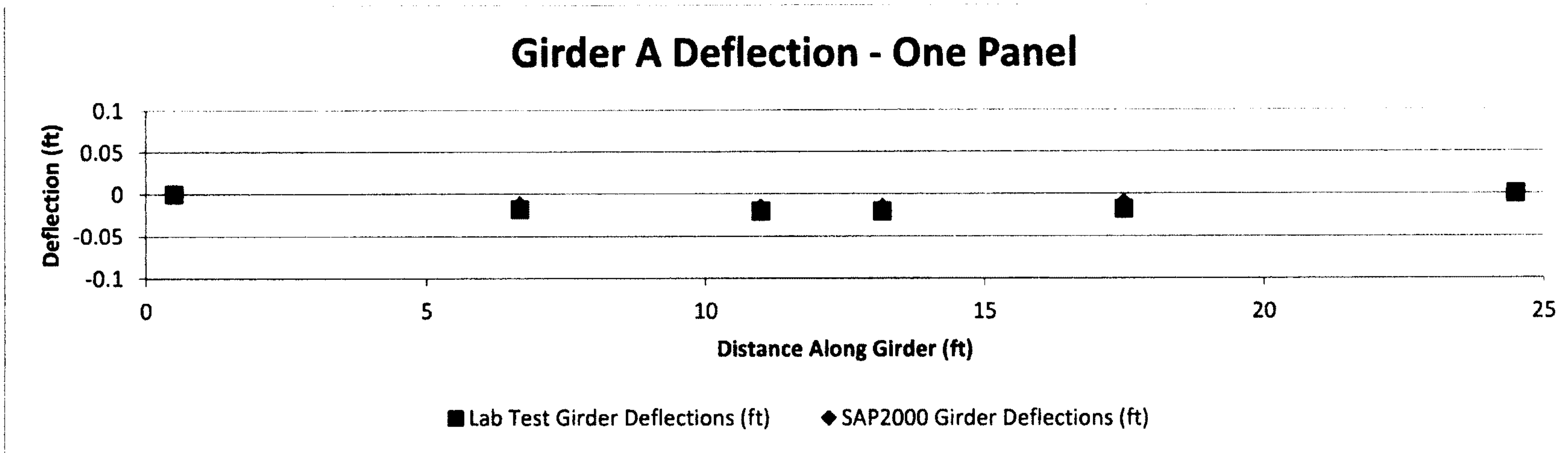
Trial 4 Girder Deflection Graphs

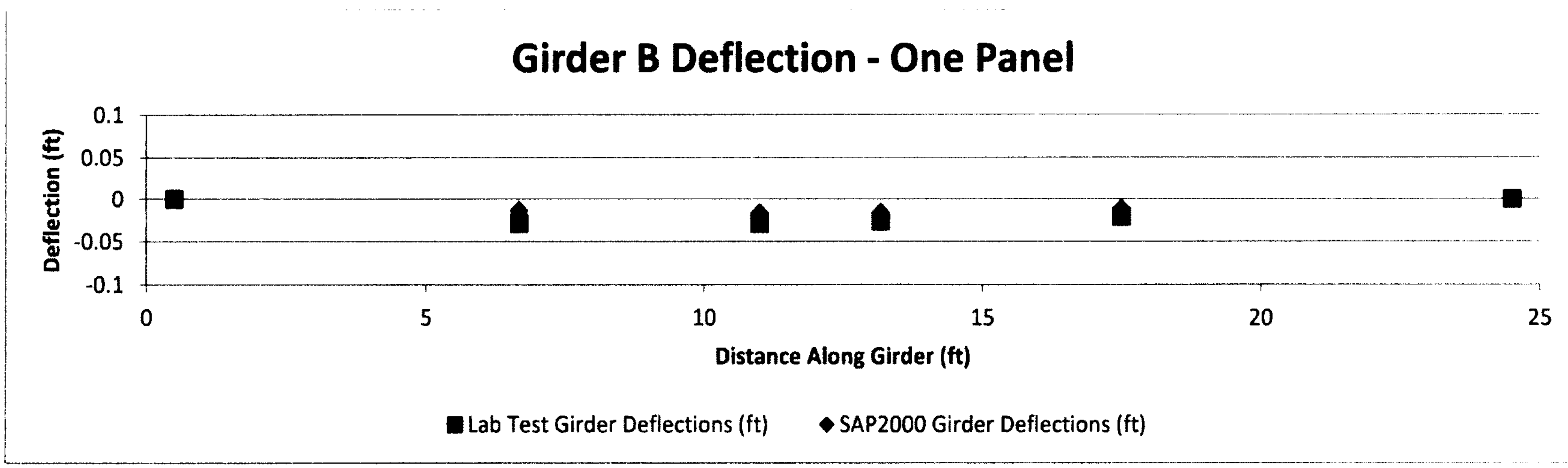


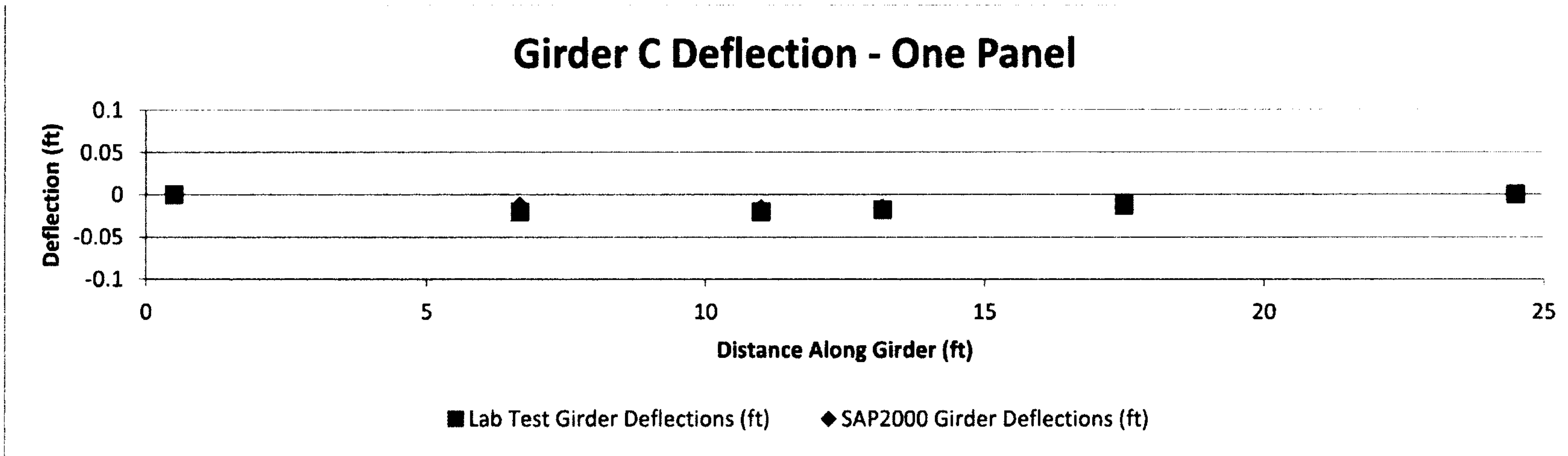



Girder C Unloaded Elevation
y = -4E-07x4 + 2E-05x3 - 0.0005x2 + 0.0066x + 2.3093
R2 = 0.9998
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
4th Order Polynomial Curve


Girder D Unloaded Elevation
y = -8E-08x4 + 7E-06x3 - 0.0002x2 + 0.0005x + 2.38
R2 = 0.9999
Elevation (ft)
3
2.8
2.6
2.4
2.2
2
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
4th Order Polynomial Curve


Girder A Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
-0.1
0
5
10
15
20
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder C Deflection - One Panel
Deflection (ft)
0.1
0.05
0
-0.05
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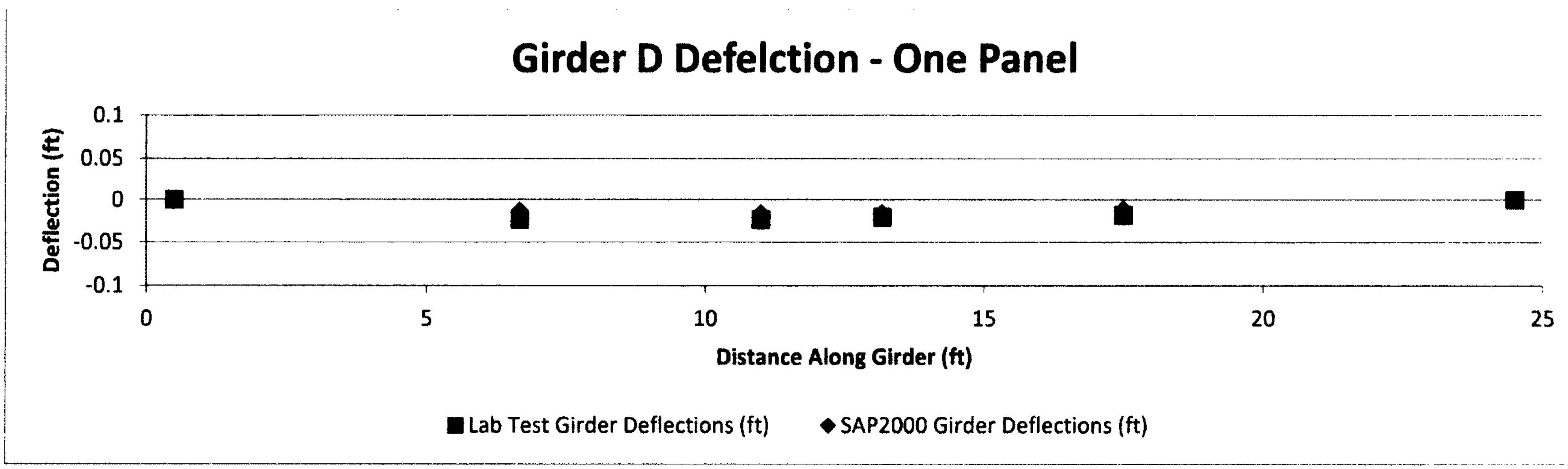
25
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Defelction - One Panel
Deflection (ft)
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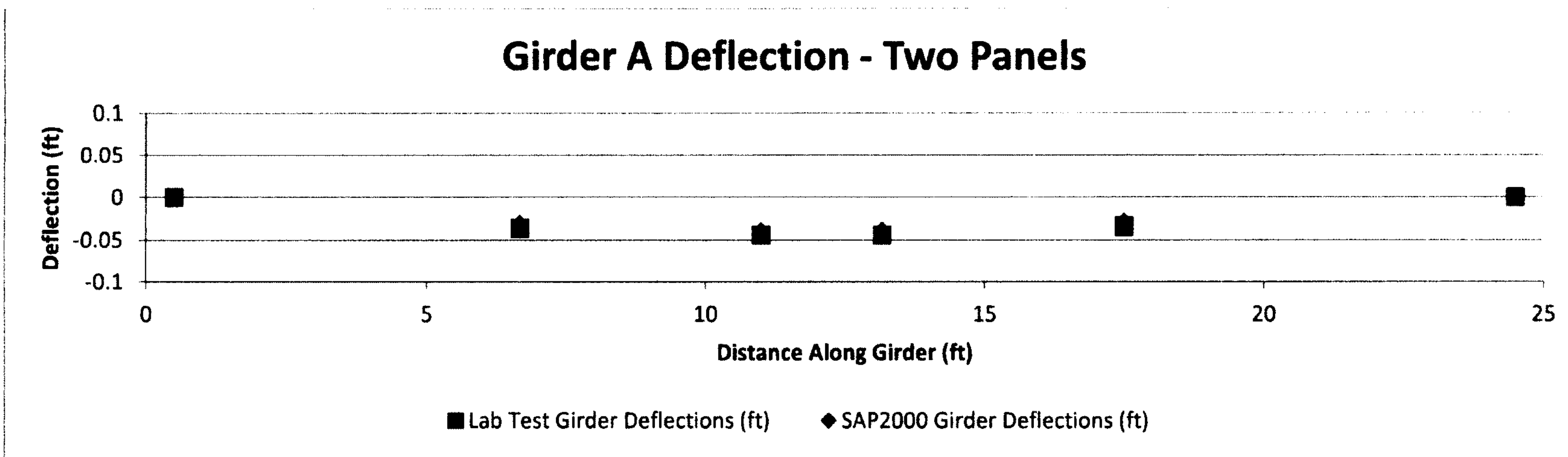
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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Deflection (ft)
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0.05
0
-0.05
-0.1
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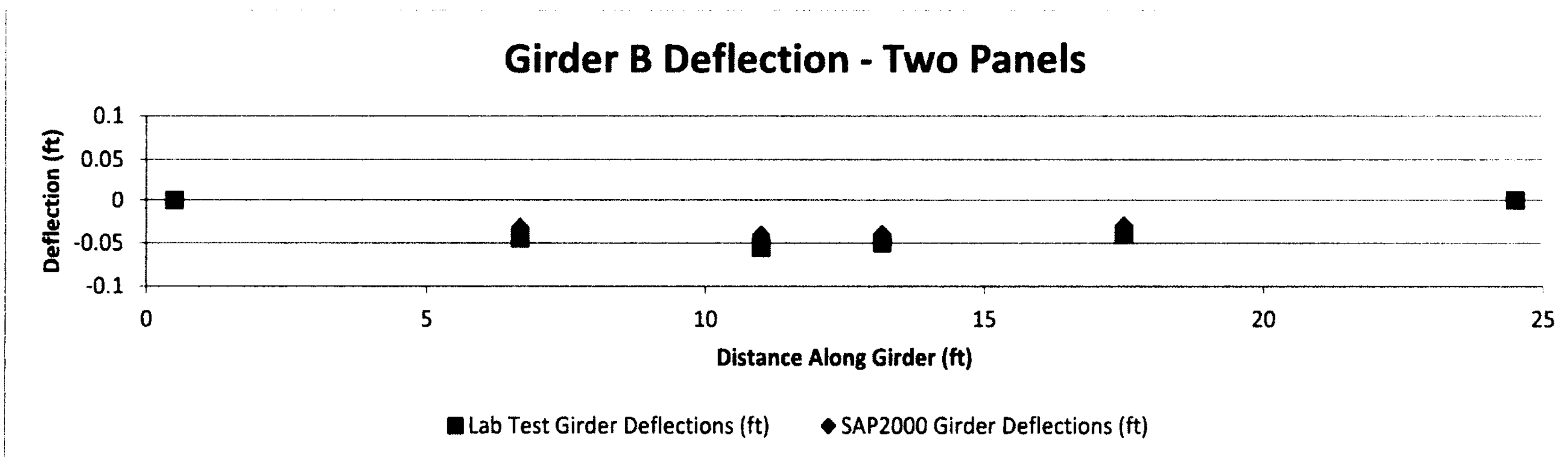
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - Two Panels
Deflection (ft)
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-0.1
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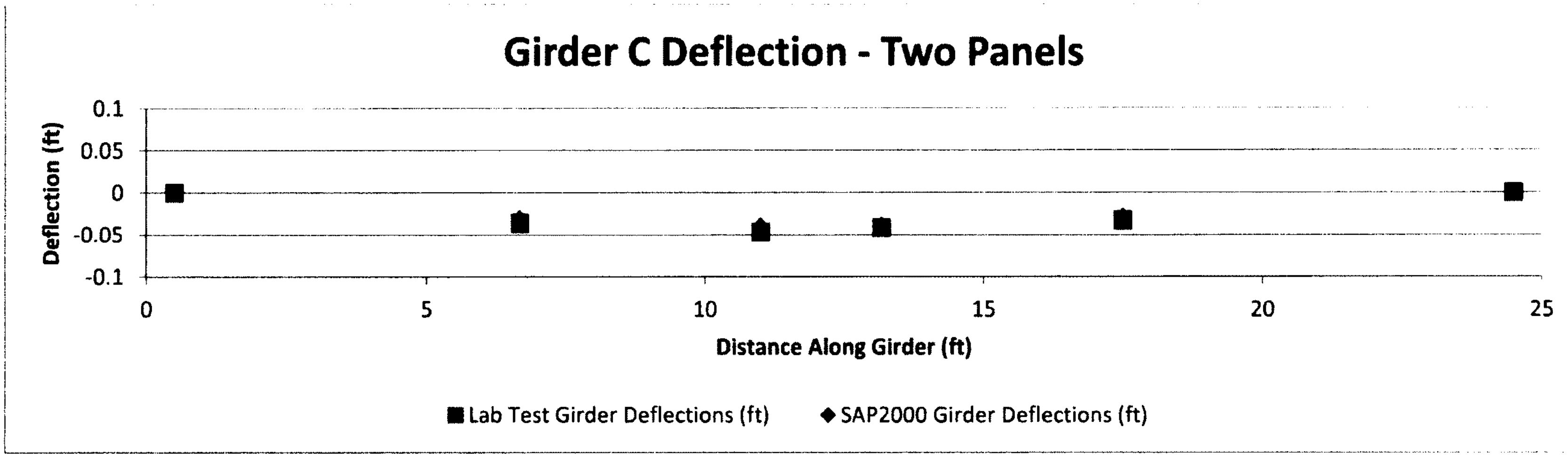
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder C Deflection - Two Panels
Deflection (ft)
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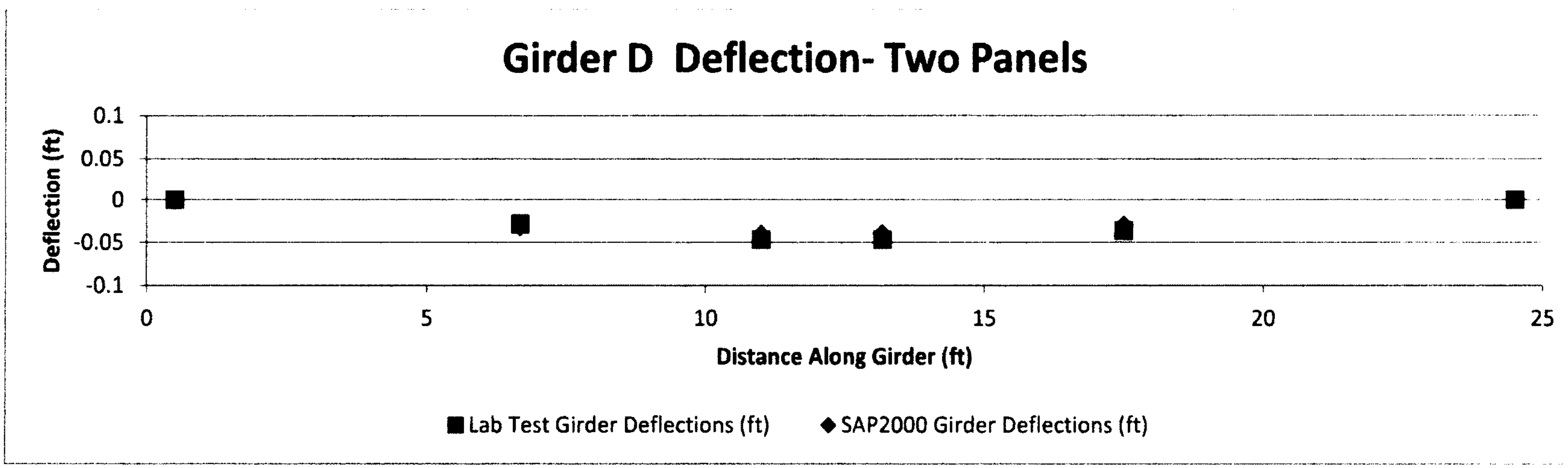
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder D Deflection- Two Panels
Deflection (ft)
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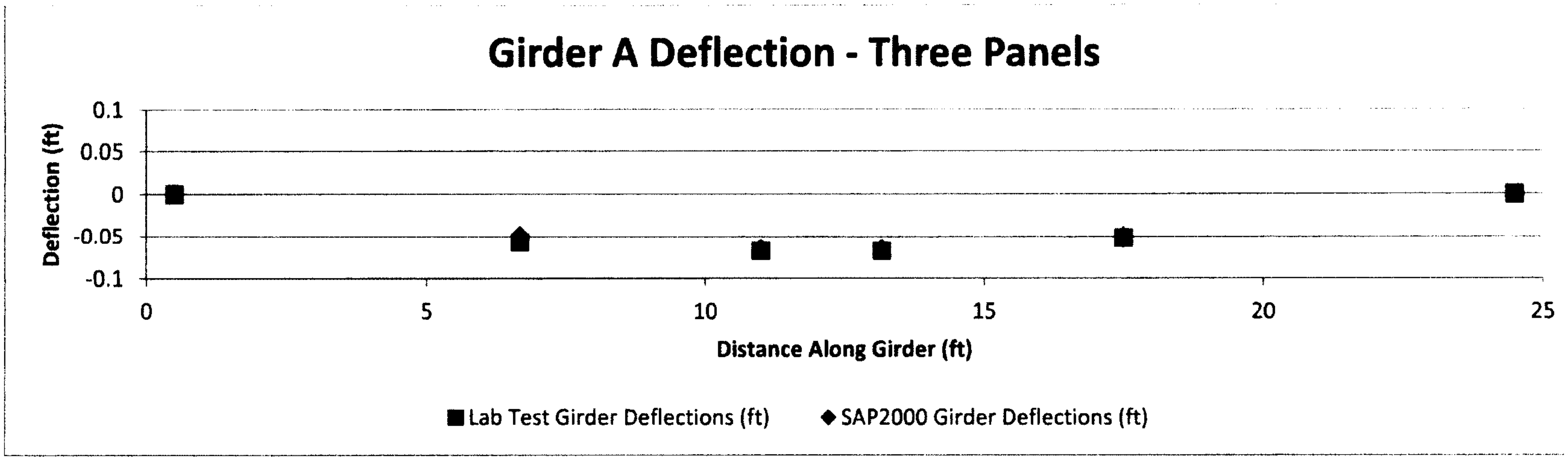
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)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Deflection (ft)
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-0.05
-0.1
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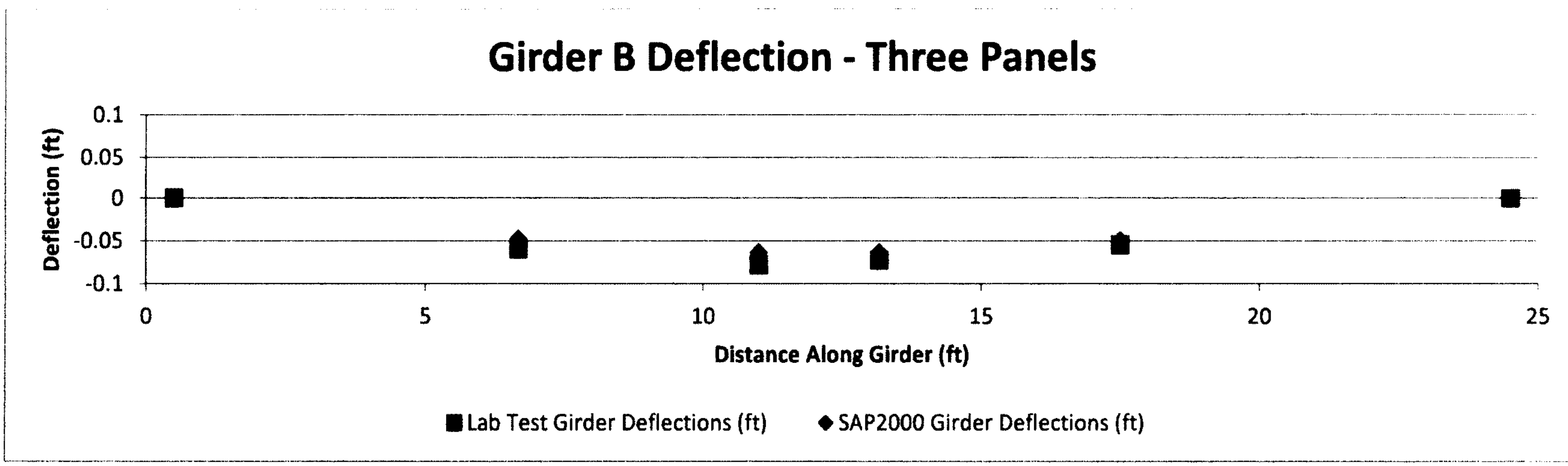
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - Three Panels
Deflection (ft)
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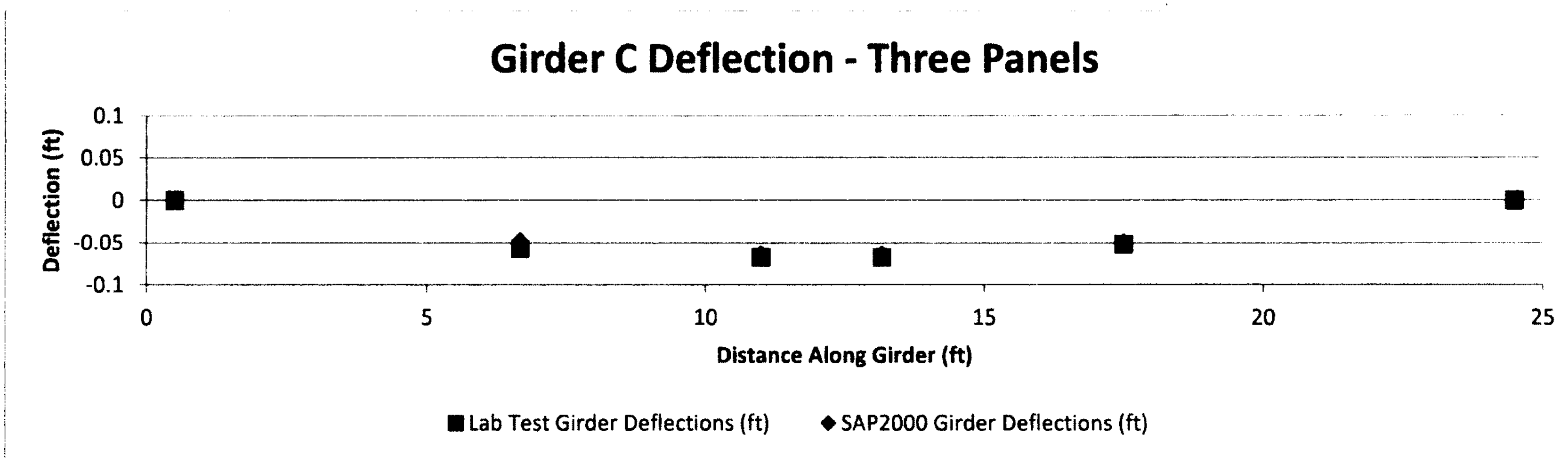
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)



Girder C Deflection - Three Panels
Deflection (ft)
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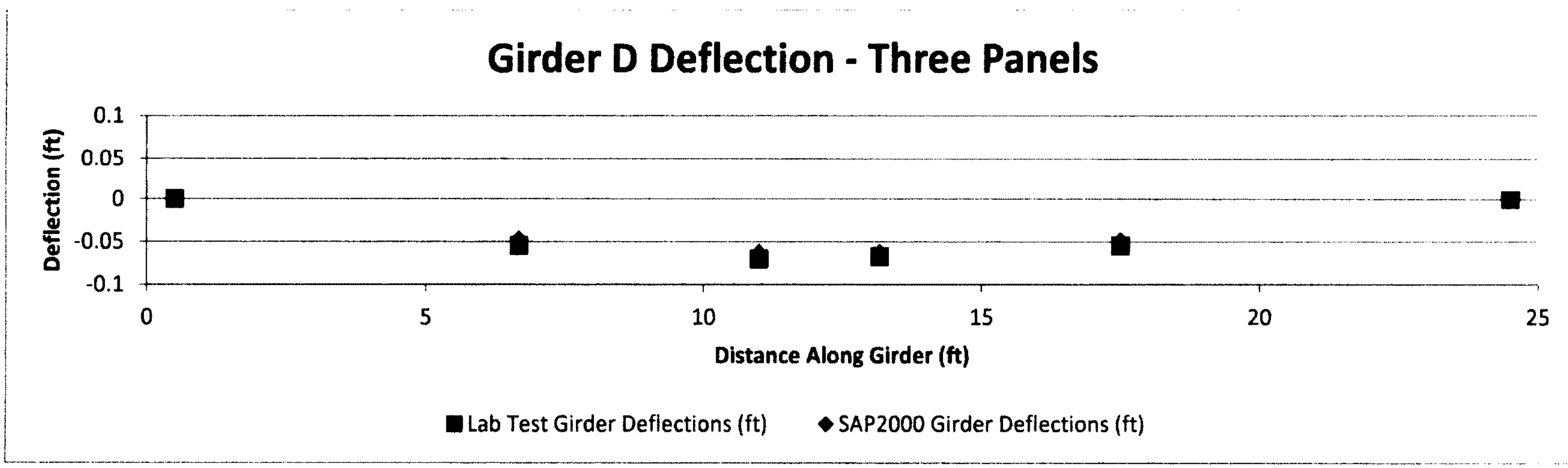
SAP2000 Girder Deflections (ft)



Girder D Deflection - Three Panels
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Distance Along Girder (ft)
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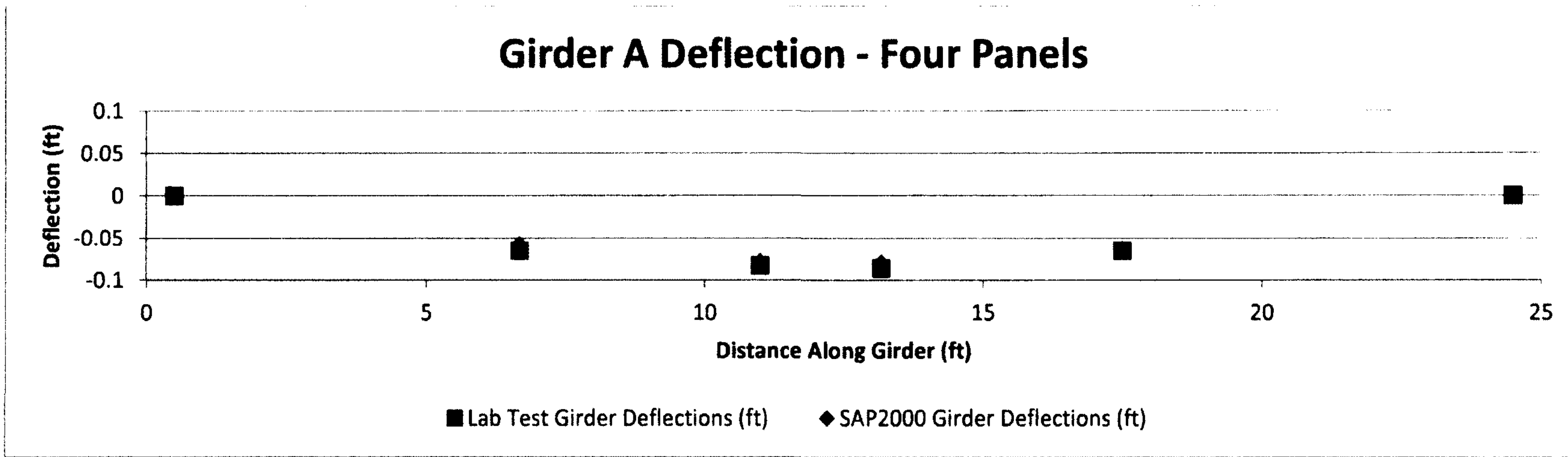
SAP2000 Girder Deflections (ft)


Girder A Deflection - Four Panels
Deflection (ft)
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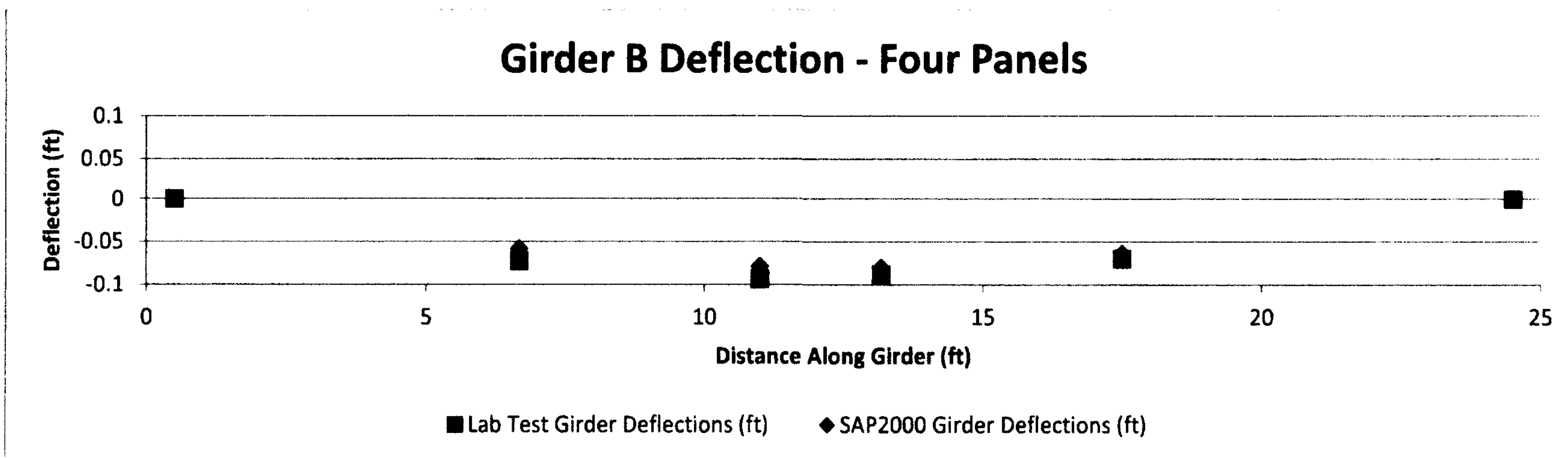
Distance Along Girder (ft)
Lab Test Girder Deflections (ft)
SAP2000 Girder Deflections (ft)


Girder B Deflection - Four Panels
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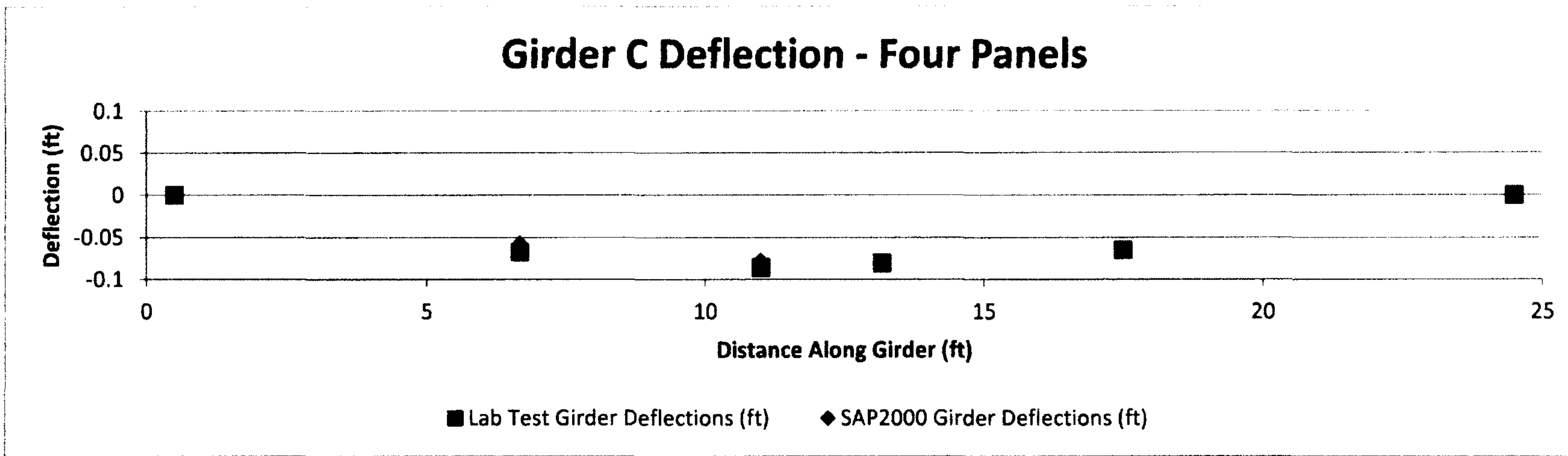
Girder C Deflection - Four Panels
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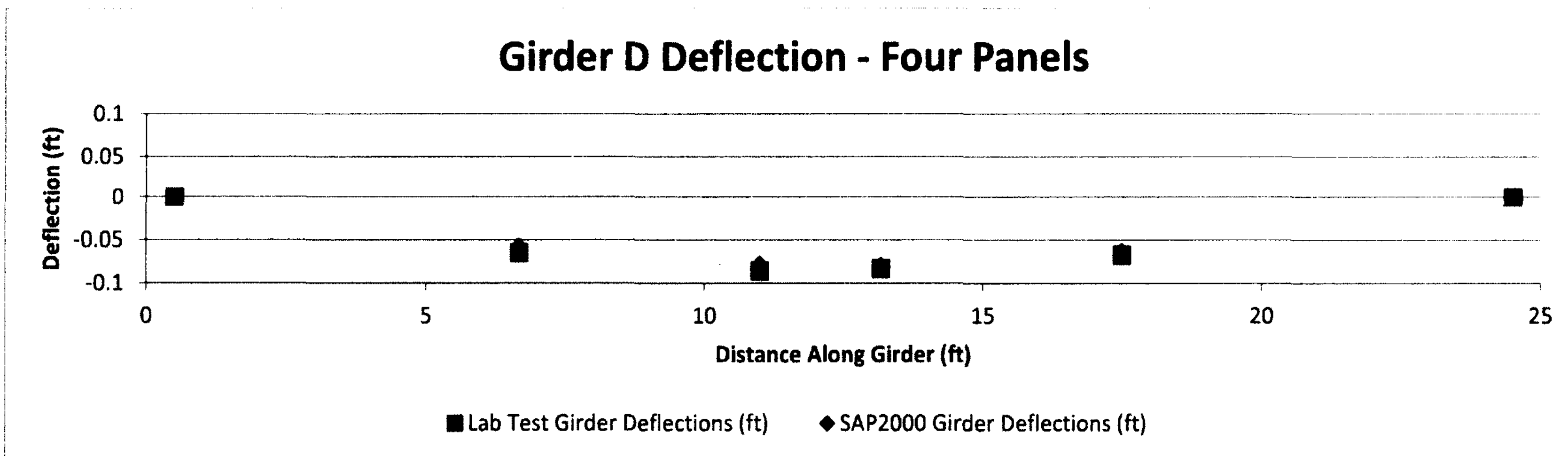
Girder D Deflection - Four Panels
Deflection (ft)
0.1
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-0.1
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SAP2000 Girder Deflections (ft)

APPENDIX D

Sikadur 31, Hi-Mod Gel Product Information

Product Data Sheet
Edition 5.5.2011
Sikadur 31, Hi-Mod Gel

Sikadur® 31, Hi-Mod Gel (1:1 Mix Ratio)

High-modulus, high-strength, structural, epoxy paste adhesive

Description	Sikadur 31, Hi-Mod Gel, is a 2-component, 100% solids, solvent-free, moisture-tolerant, high-modulus, high-strength, structural epoxy paste adhesive. It conforms to the current ASTM C-881 Types I and IV, Grade-3, Class-B/C and AASHTO M-235 specifications.
Where to Use	■ Structural bonding of concrete, masonry, metals, wood, etc. to a maximum glue line of ⅛ in. (3 mm) ■ Grout bolts, dowels, and pins ■ Seals cracks and around injection ports prior to pressure-injection grouting ■ Interior, vertical, and overhead repair of concrete as an epoxy mortar binder ■ As a pick-proof sealant around windows, doors, lock-ups etc. inside correctional facilities
Advantages	■ Meets physical requirements of ASTM C-881 Types I, II & IV, Grade 3, Classes B & C ■ Suitable for potable water contact, meets NSF/ANSI Standard 61 ■ Excellent adhesion to concrete, masonry, metals, wood, and most structural materials ■ Paste consistency ideal for vertical and overhead repair of concrete ■ Fast-setting and strength-producing adhesive ■ Convenient easy mix ratio A:B = 1:1 by volume

Typical Data *(Material and curing conditions @ 73°F (23°C) and 50% R.H.)*

RESULTS MAY DIFFER BASED UPON STATISTICAL VARIATIONS DEPENDING UPON MIXING METHODS AND EQUIPMENT, TEMPERATURE, APPLICATION METHODS, TEST METHODS, ACTUAL SITE CONDITIONS AND CURING CONDITIONS.

Shelf Life	2 years in original, unopened containers
Storage Conditions	Store dry at 40°-95°F (4°-35°C). **Condition material to 65°-85°F (18°-29°C) before using.**
Color	Gray
Mixing Ratio	Component 'A' : Component 'B' = 1:1 by volume
Consistency	Non-sag paste
Pot Life	Approximately 60 minutes @ 73°F (500 gram mass)
Tack-Free Time	1.5 - 2.5 hours at 30 mils. thick

Tensile Properties (ASTM D-638)
7 day Tensile Strength 3,300 psi (22.7 MPa)
Elongation at Break 0.9 %

Flexural Properties (ASTM D-790)
7 day Flexural Strength (Modulus of Rupture) 6,100 psi (42.0 MPa)
Tangent Modulus of Elasticity in Bending 1.67 X 10^6 psi (11,520 MPa)

Shear Strength (ASTM D-732) **7 day** Shear Strength 4,600 psi (31.7 MPa)

Bond Strength (ASTM C-882)
Hardened Concrete to Hardened Concrete:
2 day (dry cure) 2,200 psi (15.2 MPa)
2 day (moist cure) 2,400 psi (16.5 MPa)
14 day (moist cure) 2,900 psi (20.0 MPa)
Hardened Concrete to Steel:
2 day (dry cure) 2,900 psi (20.0 MPa)

Tensile Bond Strength (Pull-off Method, Dyna, ASTM C-1583-04)
2 day 420 psi (2.9 MPa)

Heat Deflection Temperature (ASTM D-648) **7 day** (Fiber Stress Loading = 264 psi) 135°F (57°C)

Water Absorption (ASTM D-570) **24 hour** 0.07%

Compressive strength (ASTM D-695) psi (MPa)

	40°F (4°C)* **	**73°F (23°C)* ****	**90°F (32°C)* ****
2 hour	-	-	450 (3.1)
4 hour	-	800 (5.5)	10,500 (72.4)
8 hour	-	8,500 (58.6)	12,200 (84.1)
16 hour	700 (4.8)	10,500 (72.4)	13,000 (89.6)
1 day	6,000 (41.4)	13,000 (89.6)	15,000 (103.4)
3 day	11,000 (75.8)	14,000 (96.5)	16,000 (110.3)
7 day	12,900 (88.9)	15,000 (103.4)	16,000 (110.3)
14 day	13,500 (93.0)	15,400 (106.1)	16,000 (110.3)
28 day	14,000 (96.5)	16,000 (110.3)	16,000 (110.3)

Compressive Modulus of Elasticity (ASTM D-695) **7 day** 7.95 X 10^5 psi (5,485 MPa)

* Material cured and tested at temperatures indicated

** See Limitations section for further information

Coverage 1 gal yields 231 cu in (3,785 cm^3) of epoxy paste adhesive 1 gal (3 8 L) mixed with 1 gal (3 8 L) by loose volume of oven-dried aggregate yields approximately 346 cu in (5,670 cm^3) of epoxy mortar

Packaging 1 gal and 3 gal (11 4 L) units

How to Use

Surface Preparation Surface must be clean and sound It may be dry or damp, but free of standing water Remove dust, laitance, grease, curing compounds, impregnations, waxes, and any other contaminants
Preparation Work: Concrete - Should be cleaned and prepared to achieve a laitance and contaminant free, open textured surface by blastcleaning or equivalent mechanical means
Steel - Should be cleaned and prepared thoroughly by blastcleaning

Mixing **Pre-mix each component** Proportion 1 part Component 'B' to 1 part Component 'A' by volume into a clean pail Mix thoroughly for 3 minutes with Sika paddle on low-speed (400- 600 rpm) drill until uniform in color Mix only that quantity which can be used within its pot life Prior to mixing, material should be conditioned to 65°-85°F (18°-29°C) To prepare an epoxy mortar, slowly add up to 1 part, by loose volume of an oven-dried aggregate, to 1 part of the mixed Sikadur 31, Hi-Mod Gel, and mix until uniform in consistency

Application **As a structural adhesive** - Apply the neat mixed Sikadur 31, Hi-Mod Gel to the prepared substrates. Work into the substrate for positive adhesion Secure the bonded unit firmly into place until the adhesive has cured Glue line should not exceed 1/8-in (3 mm)
To seal cracks for injection grouting - Place the neat mixed material over the cracks to be pressure injected and around each injection port Allow sufficient time to set before pressure injecting
For interior vertical and overhead patching - Place the prepared mortar in void, working the material into the prepared substrate, filling the cavity Strike off level Lifts should not exceed 1-in (25 mm)
As a pick-proof sealant - Use automated or manual method Apply an appropriate size bead of material around the area being sealed Seal with neat Sikadur 31, Hi-Mod Gel

Limitations

- **THE NTSB HAS STATED THAT THIS PRODUCT IS APPROVED FOR SHORT TERM LOADS ONLY AND SHOULD NOT BE USED IN SUSTAINED TENSILE LOAD ADHESIVE ANCHORING APPLICATIONS WHERE ADHESIVE FAILURE COULD RESULT IN A PUBLIC SAFETY RISK. CONSULT A DESIGN PROFESSIONAL PRIOR TO USE.**
- Components of original 2 1 mix ratio formulation of Sikadur 31, Hi-Mod Gel cannot be cross-mixed with components of Sikadur 31, Hi-Mod Gel (NEW 1 1 Mix Ratio) formulation
- Minimum substrate and ambient temperature 40°F (4°C)
- Do not thin Solvents will prevent proper cure
- When preparing an epoxy mortar, use oven-dried aggregate only
- Maximum epoxy mortar thickness is 1 in (25 mm) per lift
- Epoxy mortar is for interior use only Material is a vapor barrier after cure
- Minimum age of concrete must be 21-28 days, depending upon curing and drying conditions, for mortar applications
- Porous substrates must be tested for moisture-vapor transmission prior to mortar applications
- Not for sealing cracks under hydrostatic pressure
- Not an aesthetic product Color may alter due to variations in lighting and/or UV exposure

WARNING **Component 'A' - IRRITANT, SENSITIZER.** Contains epoxy resin, silica, and calcium carbonate Causes eye irritation. May cause skin/respiratory irritations Prolonged and/or repeated contact with skin may cause allergic reaction/sensitization Harmful if swallowed **Deliberate concentrations of vapors for purposes of inhalation is harmful and can be fatal.**
Component 'B' - CORROSIVE, SENSITIZER, IRRITANT. Contains Amines, silica quartz (sand), and calcium carbonate. Contact with skin and eyes causes severe burns. Causes eye/skin/respiratory irritation. Prolonged and/or repeated contact may cause allergic reaction/sensitization. Harmful if swallowed **Deliberate concentrations of vapors for purposes of inhalation is harmful and can be fatal.**
Cured material, if sanded, may result in exposure to a chemical known to the State of California to cause cancer

First Aid **Eyes** - Hold eyelids apart and flush thoroughly with water for 15 minutes **Skin** - Remove contaminated clothing Wash skin thoroughly for 15 minutes with soap and water **Inhalation** - Remove person to fresh air **Ingestion** - Do not induce vomiting Contact a physician **In all cases, contact a physician immediately if symptoms persist.**

Handling & Storage Avoid direct contact with eyes and skin. Wear chemical resistant gloves/goggles/clothing. Avoid breathing vapors Use with adequate general and local exhaust ventilation Use a properly fitted NIOSH approved respirator Wash thoroughly after handling product Remove contaminated clothing and launder before reuse Store product in a closed container in a cool, dry place

Clean Up	Avoid contact. Wear chemical resistant clothing/gloves/goggles. In absence of adequate ventilation, use a properly fitted NIOSH respirator. Uncured material can be removed with solvent. Follow solvent manufacturer's instructions for use and warnings. Cured material (when Component 'A' combined with Component 'B') can only be removed mechanically. In case of spill, ventilate area and contain spill. Collect with absorbent material. Dispose of in accordance with current, applicable local, state and federal regulations.

Emecole 455 Product Information

EMECOLE, INC.
P.O. Box 7486 Romeoville, IL 60446
800-844-2713 815-372-2493
www.emecole.com

EMECOLE 455 ADHESIVE SYSTEM (Fast and Slow)

GENERAL DESCRIPTION

Emecole 455 Fast and Slow adhesives are used for bonding, sealing, and repairing a wide range of properly prepared substrates including difficult-to-adhere-to plastics, metals and concrete. The only differences between the two materials are the working and cure times. These high strength, two-part, room temperature curing adhesive systems are resistant when exposed to elevated temperatures, moisture, fuels, and most solvents and chemicals. Emecole 455 Fast is recommended for use as a concrete crack surface sealer (and to secure injection ports) and is especially suited for shorter cracks. The speed of cure minimizes wasted applicator down time prior to injection. The adhesive has outstanding cured strength. It is not recommended when removal is required. It can also be used as a blow hole repair material and fast setting surface port adhesive.

TYPICAL COMPONENT PROPERTIES

	Prepolymer	Curative
Viscosity (cps)	15000	20-40000
Ratio by weight	1.08	1.00
Ratio by volume	1	1
Color	White	Gray
	455 Fast	**455 Slow**
Nominal working time	3-5 minutes	35 minutes
Nominal injection time	10-20 minutes	2 hours

PACKAGING

Dual Cartridges

APPLICATION

Remove black tips of cartridges, attach static mixer (if desired) and retaining nut to cartridges. Place cartridge in carriage of tool. Allow 1-2 ounces of material to flow out of cartridge to equalize system. Dispense at pressure giving desired output. Small amounts may also be dispensed directly from cartridges (without a static mixer) and hand stirred prior to use. If cartridge not emptied, remove static mixers and replace black tips for next use. Repeat steps if necessary.

CLEAN UP

In general, should be a moisture-free solvent. Most effective is methylene chloride, followed by MIBK. If above is not acceptable, use less efficient solvents such as mineral spirits or DOP.

WARRANTY

Recommendations concerning the performance or use of this product are based upon independent test reports believed to be reliable. If the product is proven to be defective, at the option of the Manufacturer, it will be either replaced or the purchase price refunded. The Manufacturer will not be liable in excess of the purchase price. The user will be responsible for deciding if the product is suitable for his application and will assume all risk associated with the use of the product. This warranty is in lieu of any other warranty expressed or implied, including but not limited to an implied warranty of merchantability or an implied warranty of fitness for a particular use.